(1)

AD-A234 417

NPS

RESILIENT PERFORMANCE OF CONTROLLED DENSITY FILL IN UTILITY TRENCH EXCAVATIONS

CETS 600
Graduate Research Presented for the Master of Science in Civil Engineering
Department of Civil Engineering
University of Washington

by
Darin V. Lasater

21 September 1990

RESILIENT PERFORMANCE OF CONTROLLED DENSITY FILL IN UTILITY TRENCH EXCAVATIONS

CETS 600
Graduate Research Presented for the Master of Science in Civil Engineering
Department of Civil Engineering
University of Washington

by
Darin V. Lasater

21 September 1990

Per Form 50

A-1

ABSTRACT

Controlled Density Fill (CDF) is a ready-made mix of sand, fly ash, cement, and water that, when used as backfill, flows into excavated cavities, completely filling all voids. It can be used anywhere conventional soil or aggregate backfill is used. Its advertised advantages are numerous including speed, cost, and performance. This study concentrates on CDF's performance in utility trenches as a flexible pavement subgrade backfill material where careful engineering consideration must be given in determining material properties for pavement design and analysis.

Since resilient modulus testing is the most accurate method of determining pavement subgrade suitability for soils, these tests were conducted on CDF cylinders. Moduli were compared with those of typical subgrade soils.

Subgrades with resilient modulus values greater than 15,000 psi are considered "excellent" materials. While subgrades under Washington highways averaged 19,300 psi, CDF with 40 lbs/CY of cement averaged 41,400 psi. CDF with 30 lbs/CY of cement averaged only 11,700 psi. No plastic deformation (settlement) problems were encountered after 612,000 equivalent single-axle loads with the 40 lbs/CY mix. When combined with other advantages including economy, CDF appears to be a viable subgrade material.

TABLE OF CONTENTS

INTRODUCTION

General.

Conventional Trench Backfill - Traditionally, when repairing or installing utility lines which are under flexible pavements, municipal specifications call for granular material compacted to 95% of standard procter to backfill the trench. This material becomes the new subgrade; part of the pavement structure. There are obvious problems with backfilling this kind of trench. Everyone has heeded the roughness of the pavement depression left as a marker from a utility cut. Compaction requires tight quality control and inspection. Inadequate compaction can result from lifts that are too thick, insufficient compactive effort, incorrect moisture content, or improper compaction equipment. When utilities are added to the trench, compaction becomes very difficult to achieve around the pipe area without damaging it. Additionally, voids can be created at the edge of the trench under the pavement where the subgrade material falls away from the vertical cut. If the pavement isn't cut away so that compaction equipment can compact the fill, an overbreak void is created which later settles causing pavement distresses.

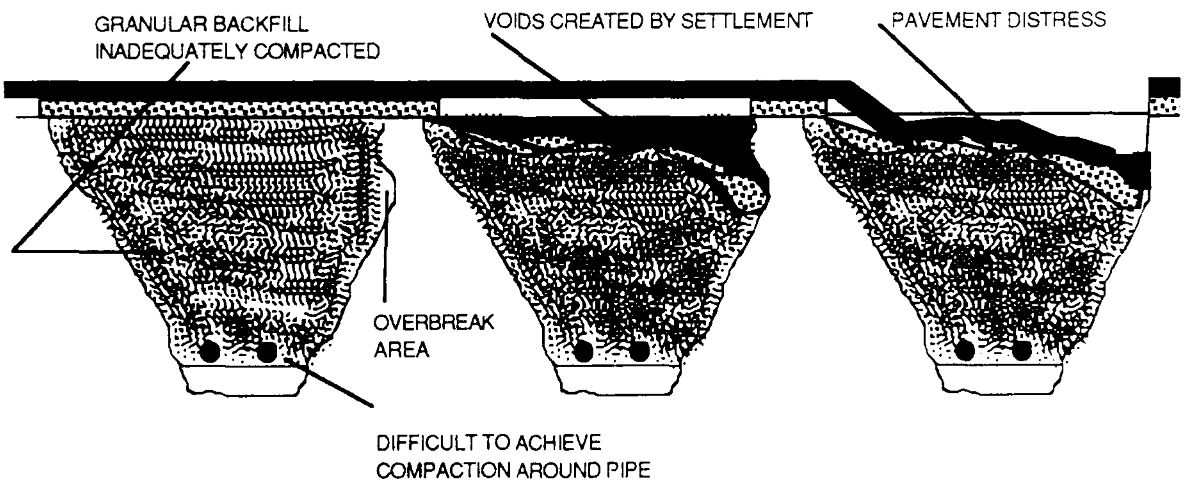


Figure 1 - Inadequately Compacted Fill Leads to Damaged Pavement

"Estimates indicate that past practices involving the number and quality of [utility-cut] restorations reduced the average pavement life expectancy by 8 to 10 years."[1] Time is money to contractors and proper subgrade compaction takes time. That is why generally the job is rushed and inadequate compaction results. Conventional fill and compaction is labor and equipment intensive which adds to the overall cost of the project above that of the fill material cost alone.

Controlled Density Fill and its Uses - Controlled Density Fill (CDF), as its currently being marketed, is a recipe fill material consisting of sand and fly ash, stabilized with cement, and mixed with water. Although seemingly a fairly recent addition to the construction industry, it has actually been around since 1974 when it was introduced by Detroit Edison and the Kuhlman Corp. as "K-Krete."[2] It has since been produced under a variety of other trade names including Flowable Fill, Flowable Fly Ash, Lean Mix Backfill, Flowable Compacting Fill (FOF), Ready-Mixed Flowable Fill (RFF), Fillcrete, Tru-crete, Flo-fill, lean concrete trench backfill, and unshrinkable fill. The product being marketed locally by Pozzolanic[3] (processors and distributors of fly ash) and distributed by ready-mix concrete producers such as Stoneway Concrete[4] and Associated Sand and Gravel[5] seems to be a viable alternative to conventional backfill. The American Concrete Institute (ACI Committee 229) will soon release a state-of-the-art report on Control Density Fill under the name Controlled Low Strength [concrete] Materials (CLSM). Other than that forthcoming report, there is little unbiased technical or informational data on the substance.

The opportunities for CDF use include:

- Sewer, electrical duct, or other utility trench backfill.
- Abandoned tank, manhole, or pipe fill
- Culvert backfilling
- Pipe bedding
- Pavement subgrade fill
- Temporary slabs for military or construction equipment laydown areas

•Foundation subbase and backfill

•Bridge abutment and retaining wall backfill

•Or anywhere else conventional backfill is used

In the course of this research, the primary interest in CDF's use was as a utility trench backfill material and its performance as a subgrade under pavements when a cut to repair / replace utilities must be made.

Structurally, CDF is somewhere between a concrete and a soil. It's placed at a fluid consistency and requires no compaction to achieve its density and strength. Its load capacity is advertised to be typically much stronger than compacted soil yet still excavatable by conventional means. Its makers claim "the ease of placement, high density, and greater strength of CDF makes it superior to standard backfill. It can be used wherever soil backfill is used and in most cases where granular backfill is used." [6]

CDF Advantages

The following potential advantages over conventional backfill have been compiled and paraphrased from promotional materials:

Controlled Strength /Density and Quality Control - You can select the desired density and compressive strength for your project, CDF ranges from 90 to 150 lbs/ft^3 and 0 to 1600 psi.[7] It is then custom mixed and delivered by ready-mix concrete companies.

Workability - Because of the relatively high water content and the "ball bearing" action of the fly ash pozzolan, the mix flows like a fluid and distributes itself around pipes and footings without vibration. It can be placed in the same manner as concrete (including pumping) but later excavated easily with a pick and shovel, backhoe, or air spade, in the same manner as compacted soil. The resulting trench can be cut without caving in or running. This is of particular importance to municipalities who must have access to underground utilities for maintenance.

Controlled Bearing Capacity and Reduced Settlement - CDF prevents cracks and depressions in pavement by reducing trench backfill settlement. Because CDF flows and

distributes itself in and around all pipes and obstructions, there are no voids commonly encountered around utilities due to the difficulty compacting in congested / tight areas. It is believed there is less settlement of CDF than with conventional compacted soil also because water escapes as placement occurs. Any minor vertical shrinkage occurs within the first few hours. CDF consolidates through the bleeding process to achieve particle to particle contact when it reaches 95-100% of optimum density. Because of the ready-mix process, there is no differential settlement because the mix and shrinkage are homogeneous. When required, CDF can have unconfined compressive strengths much higher than conventional fill materials. Reduced inspections for compaction may eliminate potential contractor blunders, resulting in better quality work with less hassle for everyone, including the contractor.

Environmental Benefits - "About 42 million tons of fly ash are produced each year from coal-fired powerplants. Seventy-five to 80% of the fly ash used in construction is in roads and highways."[8] Because CDF's composition includes fly ash, its use helps mitigate disposal problems of a "non-hazardous nuisance dust"[9] by recycling the fly ash in a safe economical fashion. It requires no special sealer or containment as the particles are effectively bonded and confined within the excavation. The EPA polices section 6002 of the Resource Conservation and Recovery Act (1984) which calls for the use of waste materials by any agency receiving federal funding. To the Federal Highway Administration "that means any agency that buys more than $10,000 worth of concrete per year must remove restrictions on fly ash where technically appropriate"[10] or face complete loss of federal funding.

Economy - Since no granular fill is required, there is no backfilling labor crew, and therefore, no protective shoring is required (except for in the immediate pipe repair area.) There is no stockpiling of fill, no compaction equipment, no placing in lifts, and no pipe damage from heavy compaction equipment. No leveling is required since in its fluid state it levels itself. It can be placed in any weather and in wet trenches where backfilling soil

would be impossible since CDF displaces the water. CDF is advertised to harden enough to support normal traffic loads in about 4 hours. This benefit alone can be of great importance in municipal or military applications by reducing road or airfield down time significantly. Because the material more completely fills the trench, preventing settlement and pavement damage, maintenance costs also decrease. "Studies and field experience have shown that sand compaction which requires 4 men and 3 days can be accomplished by using [CDF] with 2 men in 3 hours!"[11] According to Mel Hitch, sales representative at Stoneway Concrete, CDF sells for between $27 and $35 per cubic yard depending on the mix and type of consumer (based on annual volume). "The inefficient use of manpower and equipment incidental to small, widely dispersed and sporadic excavations as well as problems caused by heavy traffic -- make conventional fill methods much more expensive. The true cost is difficult to estimate, especially if future maintenance costs are considered."[12] Tom Howerton, of Associated Sand & Gravel, who has been supplying CDF to the City of Everett, WA for pavement cut backfill since 1987, charges the City contractors $30 per cubic yard to site deliver the appropriate mix. He claims, in a review of past bids, the City was paying $30 per cubic yard for conventional pit run backfill projects ($10-15 material cost). The same analysis shows CDF is bid at about $40 per cubic yard - a 33% first cost increase. The City of Everett pays the additional cost on major arterial street cuts to take advantage of the short duration set time so that the road can be reopened to traffic as soon as possible.[13] In the three years of service, the City has had no problems with pavement settlement.

Purpose

The purpose of this investigation was to compare the resilient behavior (stiffness) of CDF to that of typical pavement subgrade materials. This study primarily concentrates on CDF's performance as a subgrade backfill material in utility trenches under asphalt pavements. Since resilient modulus testing is the most widely accepted method of determining pavement subgrade suitability, these tests were conducted on several CDF

cylinders. For comparison purposes, resilient modulus data for soils was compiled from a number of sources including additional lab work and a separate study conducted by the Washington State Transportation Center (TRAC - a joint venture between the two state universities and the Washington State Department of Transportation.)

BACKGROUND

Conventional Pavement Subgrades

Like a building foundation, all pavement designs must start with a consideration of the underlying soil conditions. Specific soil properties depend on many factors including mineral composition, climate, age, and method of transportation.[14] Soils for civil engineering applications are usually broken down into coarse-grained (gravels and sands) and fine-grained (silts and clays) major categories. Two standard classification systems further break down the particle size analysis for use in determining relative properties for civil engineering applications -- The Unified Soil Classification System (USCS) and the AASHTO (American Association of State Highway & Transportation Officials) Classification for Soils. Most engineers and contractors know from experience good subgrade soils and both classification systems give broad generalizations on soil suitability for pavement foundations.

Unified Soil Classification System - Casagrande proposed the Unified Soil Classification System adopted by the Army Corps of Engineers in 1942. Appendix B to the 1953 revision entitled "Characteristics of Soil Groups Pertaining to Roads and Airfields" summarized Casagrande's findings of subgrade of base course performance. "The properties desired in soils for foundations under roads and airfields and for base courses under flexible pavements are: adequate strength, good compaction characteristics, adequate drainage, resistance to frost action in areas where frost is a factor, and acceptable compression and expansion characteristics."[15] He concluded that textural classification alone was inadequate for cohesive soils. He found he could group fine-grained soils according to their liquid limit and plasticity index. The importance of plasticity in

engineering applications today is well documented. "Stiffness" was probably only considered as a secondary strength property. Table 1 summarizes Appendix B of the 1953 Corps of Engineers Unified Soil Classification System revision in a thorough graphical format.

CHARACTERISTICS PERTINENT TO ROADS AND AIRFIELDS

Major Divisions (1)	(2)	Symbol: Letter (3)	Symbol: Hatching (4)	Symbol: Color (5)	Name (6)	Value as Subgrade When Not Subject to Frost Action (7)	Value as Subbase When Not Subject to Frost Action (8)	Value as Base When Not Subject to Frost Action (9)	Potential Frost Action (10)	Compressibility and Expansion (11)	Drainage Characteristics (12)	Compaction Equipment (13)	Unit Dry Weight lb per cu ft (14)	Typical Design Values: CBR (15)	Typical Design Values: Subgrade Modulus k lb per cu in. (16)
COARSE-GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125-140	40-80	300-500
		GP		Red	Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Good	Fair to good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110-140	30-60	300-500
		GM d		Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Good	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	125-145	40-60	300-500
		GM u		Yellow		Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	115-135	20-30	200-500
		GC		Yellow	Clayey gravels, gravel-sand-clay mixtures	Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	130-145	20-40	200-500
	SAND AND SANDY SOILS	SW		Red	Well-graded sands or gravelly sands, little or no fines	Good	Fair to good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	110-130	20-40	200-400
		SP		Red	Poorly graded sands or gravelly sands, little or no fines	Fair to good	Fair	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	105-135	10-40	150-400
		SM d		Yellow	Silty sands, sand-silt mixtures	Fair to good	Fair to good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120-135	15-40	150-400
		SM u		Yellow		Fair	Poor to fair	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-130	10-20	100-300
		SC		Yellow	Clayey sands, sand-clay mixtures	Poor to fair	Poor	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-135	5-20	100-300
FINE-GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor to fair	Not suitable	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90-130	15 or less	100-200
		CL		Green	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired roller, sheepsfoot roller	90-130	15 or less	50-200
		OL		Green	Organic silts and organic silt-clays of low plasticity	Poor	Not suitable	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired roller, sheepsfoot roller	90-105	5 or less	50-100
	SILTS AND CLAYS LL IS GREATER THAN 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Not suitable	Medium to very high	High	Fair to poor	Sheepsfoot roller, rubber-tired roller	80-105	10 or less	50-100
		CH		Blue	Inorganic clays of high plasticity, fat clays	Poor to fair	Not suitable	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	90-115	15 or less	50-150
		OH		Blue	Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	80-110	5 or less	25-100
HIGHLY ORGANIC SOILS		Pt		Orange	Peat and other highly organic soils	Not suitable	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical	-	-	-

Note:

1. Column 3, division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is on basis of Atterberg limits; suffix d (e.g., GMd) will be used when the liquid limit is 25 or less and the plasticity index is 5 or less; the suffix u will be used otherwise.
2. In column 13, the equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled. In some instances, several types of equipment are listed because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.
   a. Processed base materials and other angular materials. Steel-wheeled and rubber-tired rollers are recommended for hard, angular materials with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.
   b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.
   c. Equipment size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:
      Crawler-type tractor -- total weight in excess of 30,000 lb.
      Rubber-tired equipment -- wheel load in excess of 15,000 lb, wheel loads as high as 40,000 lb may be necessary to obtain the required densities for some materials (based on contact pressure of approximately 65 to 150 psi).
      Sheepsfoot roller -- unit pressure (on 6- to 12-sq-in. foot) to be in excess of 250 psi and unit pressures as high as 650 psi may be necessary to obtain the required densities for some materials. The area of the foot should be at least 5 per cent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.
3. Column 14, unit dry weights are for compacted soil at optimum moisture content for modified AASHO compaction effort.

Table 1[16] - *The Unified Soil Classification System, Appendix B, Characteristics of Soil Groups Pertaining to Roads and Airfields* (Vicksburg, MS: U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, 1957) Table B1.

AASHTO System - The American Association of State Highway and Transportation Officials (AASHTO) system of soil classification was developed in 1928 with the 1945 revision forming the basis of today's version. This system was also based on field performance of soils as highway subgrades and is widely used. Soils with approximately equivalent load carrying capacity are grouped together into seven basic categories A-1 through A-7. In general, the best highway subgrades are rated A-1 with higher numbers designating progressively poorer performing soils. Subdivisions within the seven categories further represent relative performance as a subgrade. Table 2 summarizes the AASHTO system including each soil's general rating as a subgrade.

General Classification	Granular Materials (35% or less passing 0.075 mm)							Silt-Clay Materials (More than 35% passing 0.075 mm)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, Percent Passing											
2.00 mm (No. 10)	50 max	---	---	---	---	---	---	---	---	---	---
0.425 mm (No. 40)	30 max	50 max	51 min	---	---	---	---	---	---	---	---
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of Fraction Passing 0.425 mm (No. 40)											
Liquid Limit	---		---	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual Types of Significant Constituent Materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good							Fair to Poor			

Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure V-2).

Table 2[17] - Classification of Soils and Soil-Aggregate Mixtures

Resilient Modulus

Historically, pavement subgrade suitability has been based on static strength tests such as the California Bearing Ratio (CBR) and triaxial tests. CBR is more a relevant measure of how the material will compact and behave during the unconfined construction phase. Established primarily for crushed aggregate bases, a CBR value for subgrade soils is so low that it loses its meaning except on a relative basis. In recent years, pavement designers have been leaning more towards a mechanistic approach based on elastic layer theories which better simulates the dynamic wheel loads induced by traffic on the pavement. The resilient modulus test (AASHTO T274 - 82 (1986)) introduces the cylindrical specimen to repeated dynamic loads and confining pressures representative of field conditions. "The resilient modulus test provides a means of evaluating pavement construction materials, including subgrade soils under a variety of environmental conditions and stress states that realistically simulate the conditions that exist in pavements subjected to moving wheel loads."[18] As a result, the resilient modulus test has become the "state-of-the-art" measure of base and subgrade performance properties. "All pavement structural design procedures require a "subgrade soil" input,"[19] and mechanistic pavement and overlay design procedures specify the use of subgrade moduli.

A flexible pavement is made up of three basic layers; the asphalt concrete, the granular base course, and the subgrade soil. Since the deformation of the subgrade under a wheel load makes up a major portion of the total deformation, it is important to have a solid subgrade foundation.[20]

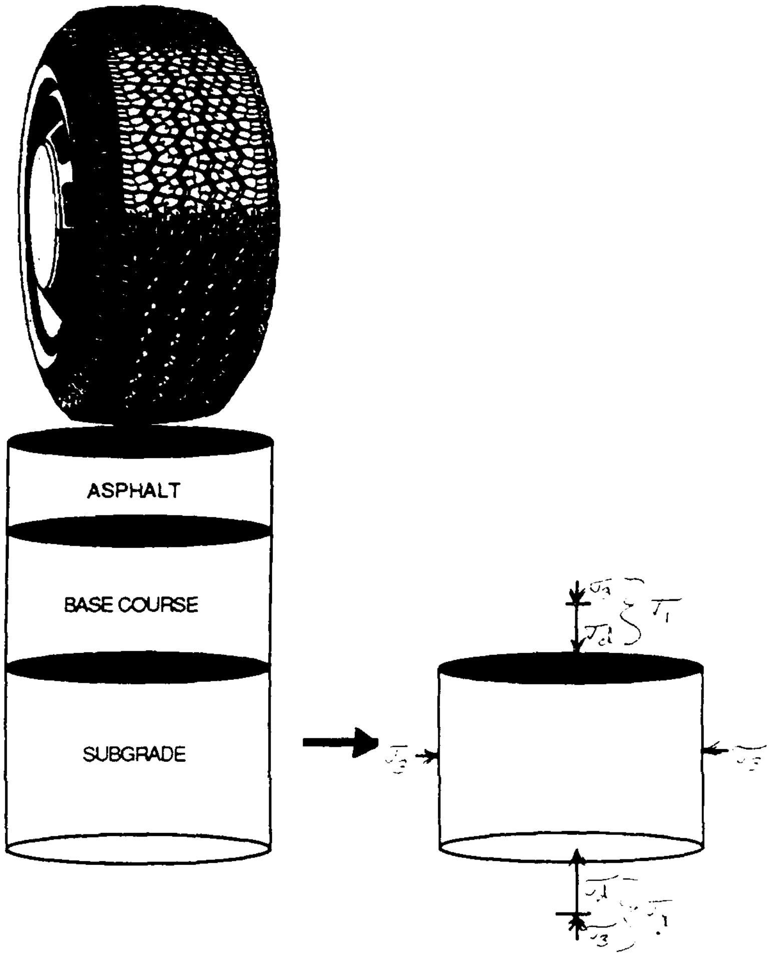


Figure 2 - Subgrade Specimen Subjected to Confining Pressures and Axial Load

Resilient modulus is defined by the following equation (For convenience, the AASHTO notation M_r is used when referring to resilient modulus):

$$M_r = (\sigma_1 - \sigma_3) / \varepsilon_{axial} = \sigma_d / \varepsilon_{axial}$$

where σ_1 = major principal stress ($\sigma_3 + \sigma_d$)

σ_3 = minor principal stress (due to confining pressure)

σ_d = principal stress difference or deviator stress ($\sigma_1 - \sigma_3$ due to applied load)

ε_{axial} = recoverable or elastic axial strain

"The soil deformation is composed of a permanent (or plastic) component, and a recoverable resilient (or elastic) component"[21] (Figure 3). It is simply a measure of the elastic stress-strain relationship (stiffness) obtained after plastic strain has been worked out with repeated ax loads (resilience). "During repeated load tests, it is observed that after a number of loading cycles, the modulus becomes approximately constant and the soil response can be assumed as elastic. The modulus at the steady soil response is defined as the resilient modulus, [M_r], and is found to occur after about 100 to 200 cycles of loading."[22]

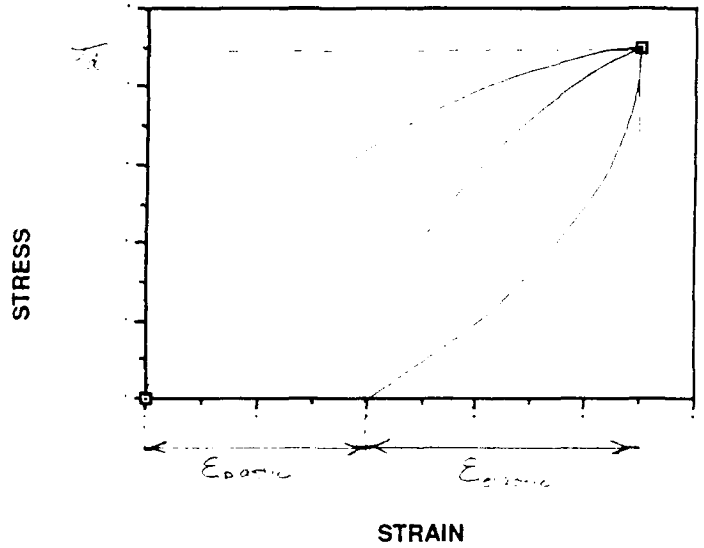


Figure 3 - Elastic Stress-Strain Relationship

The modulus value is similar to elastic modulus (E) properties of other common construction materials (Figure 4) except dynamics are introduced.

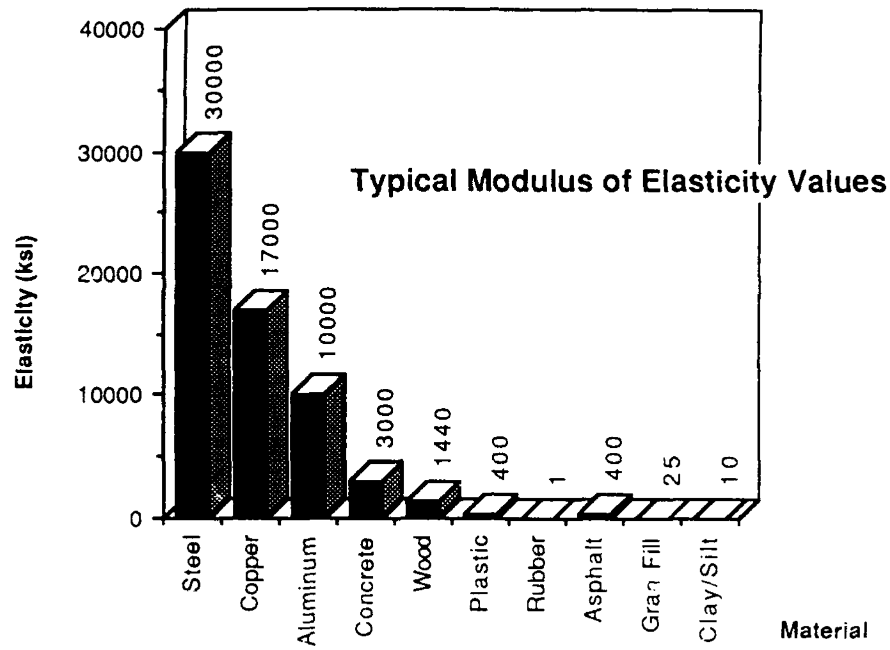


Figure 4 - Common Material Elastic Modulus (E) Values[23,24,25,26]

The AASHTO Guide for the Design of Pavement Structures recognizes resilient modulus as the definitive property used to characterize roadbed soils for the following reasons:[27]

(1) It indicates a basic material property which can be used in mechanistic analysis of multi-layered systems for predicting roughness, cracking, rutting, faulting, etc.

(2) Methods for the determination of M_r are described in AASHTO Test Method T274.

(3) It has been recognized internationally as a method for characterizing materials for use in pavement design and evaluation.

(4) Techniques are available for estimating the M_r properties of various materials in-place from non-destructive tests.

A schematic diagram of the resilient modulus apparatus is shown in Figure 5.

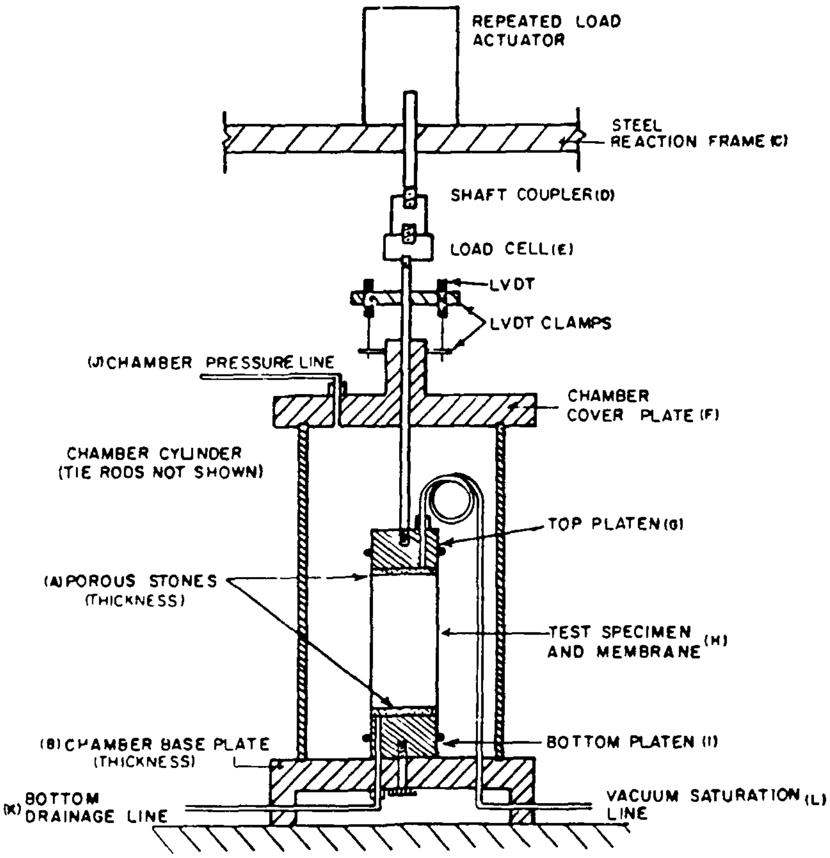


TABLE OF MEASUREMENTS (TYPICAL)

DIMENSION	A	B	C	D	E	F	G	H	I	J	K	L
METRIC, mm.	6.4	25.4	12.7	25.4	Note 1	19.1	38.1	Note 2	38.1	6.4	6.4	6.4
ENGLISH in	0.25	1.00	0.5	1.00		0.75	1.50		1.50	0.25	0.25	0.25

1 Dimension varies with manufacturer
2 Dimension varies with specimen size

Figure 5 - Schematic of Resilient Modulus Triaxial Chamber

Conversion and Regression Equations for Obtaining M_r - Although resilient moduli are recognized as <u>the</u> design values necessary for pavement subgrades, the process of obtaining these dimensions is relatively new and lengthy, and therefore can be an expensive undertaking. As mentioned, structural pavement design in the past has been based on static testing techniques such as the CBR for determining subgrade suitability.

<u>Fine-grained</u>. In the absence of more precise agency data from laboratory resilient modulus testing or non-destructive testing, AASHTO allows the following relationship to be used: M_r (psi) = 1500 X CBR.[28] They consider this direct correlation to be valid only

for fine-grained soils with a CBR of 10 or less. Another more scientific correlation, the regression equation where $M_r = K_1(\sigma_d)^{K2}$ is often used, where σ_d is the applied axial stress (deviator stress), and where K_1 and K_2 are property constants unique to the specific material (or soil type) being evaluated.[29] These constants and the resulting moduli for fine-grained soils are sensitive to moisture content and density, but they are fairly insensitive to confining pressures (uniform 3-dimensional principal stresses). Stresses in the subgrade can by approximated or roughly determined using an elastic layer analysis. The resilient moduli of cohesive clays and silts generally decrease with increasing deviator stress (K_2 is often negative)[30].

<u>Coarse-grained</u>. AASHTO uses a like correlation for granular (base) materials where E_{bs} (M_r of a base) = $K_1 Ø^{K2}$ where Ø = bulk stress = sum of the principal stresses (σ_1 + 3 * confining pressure). Unlike fine-grained soils, quality granular material generally gets stiffer as the bulk stress (Ø) increases. When the regression equations of granular materials are plotted on a log-log graph, the results typically take on the form of Figure 6.

AASHTO also allows a CBR conversion for unbound granular materials as follows:

Ø (psi)	Mr (psi)
100	740 X CBR
30	440 X CBR
20	340 X CBR
10	250 X CBR

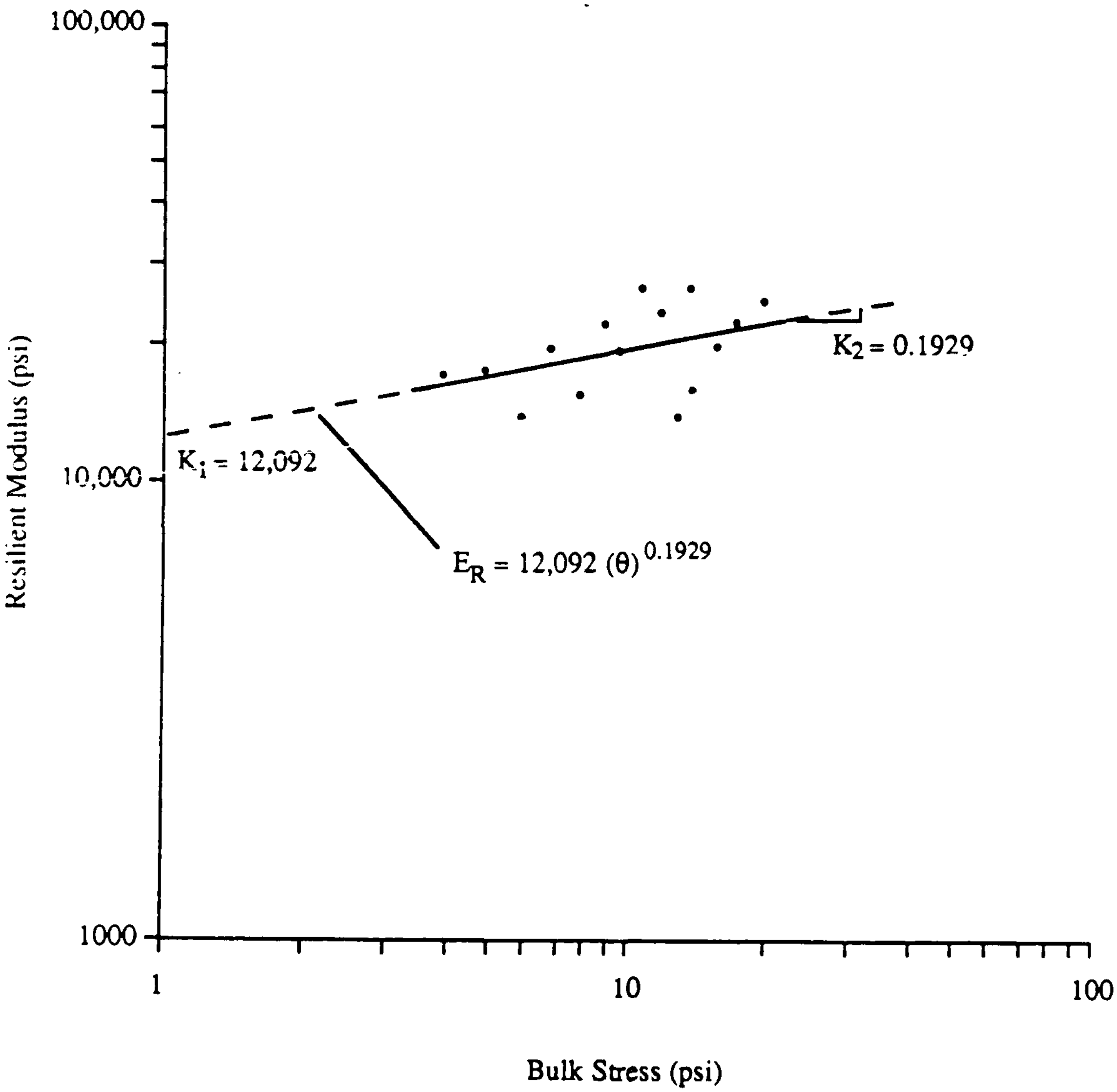


Figure 6 - Fitted Regression Line (Log Transformed) for Resilient Modulus Data[31]

Controlled Density Fill. Since CDF is neither a fine or coarse grained soil, but a stabilized material, it is unclear which (if either) regression equation best represents resilient moduli at varying bulk and deviator stresses. An attempt was made to determine if regression equations for stabilized soils (cement, lime, lime-fly ash) have been developed by agencies designing and evaluating pavements. Representatives from AASHTO, Transportation Research Board, North Carolina DOT, FHWA, and Texas DOT were contacted. Most agencies were just getting their feet wet with resilient modulus testing and have not been at it long enough to develop correlations for their materials. In the absence of more precise laboratory determined regression equations, logarithmic equations using both bulk stress and deviator stress were used for each CDF sample to determine which best represents CDF's stress sensitive behavior. A less cemented structure that derived strength from interparticle friction would be expected to be more sensitive to confining pressures than would say a block of concrete and would therefore, be better represented by a bulk stress equation.

In a study conducted on cement and lime stabilized materials[32], stabilization of sand with cement significantly increased stiffness and corresponding resilient moduli values. The resilient moduli for the stabilized materials all exceeded 15,000 psi, which is considered very good for a subgrade material. Although the stiffness of the stabilized materials increased, the values were less predictable as the stress varied. The best fit equation did not correlate well with stabilized materials (The correlation coefficient - R^2 was much lower). As expected, stabilized materials were relatively insensitive to confining pressures (better curve fit against deviator stress).

Subgrade Stresses - Since stiffness is a function of the stresses acting on the material, it is important to know what pressures are working at the subgrade level. The pavement structure can be analogized to be made up of layers of plates. Suppose the upper plate is steel, the middle layer is a sheet of plywood, and they are spanning a trench of uncompacted sand. This sand will "settle" (undergo plastic deformation) when stresses

reach its load bearing capacity. The "stiff" steel layer deflects elastically under a wheel load only slightly, protecting the plywood and sand layers from excessive stresses. The stiffer or thicker the overlying material , the more spreading of the load is experienced. As with the steel plate, the thicker a layer of asphalt concrete, the more protection (less deflection and resulting stress) is provided the layers below.

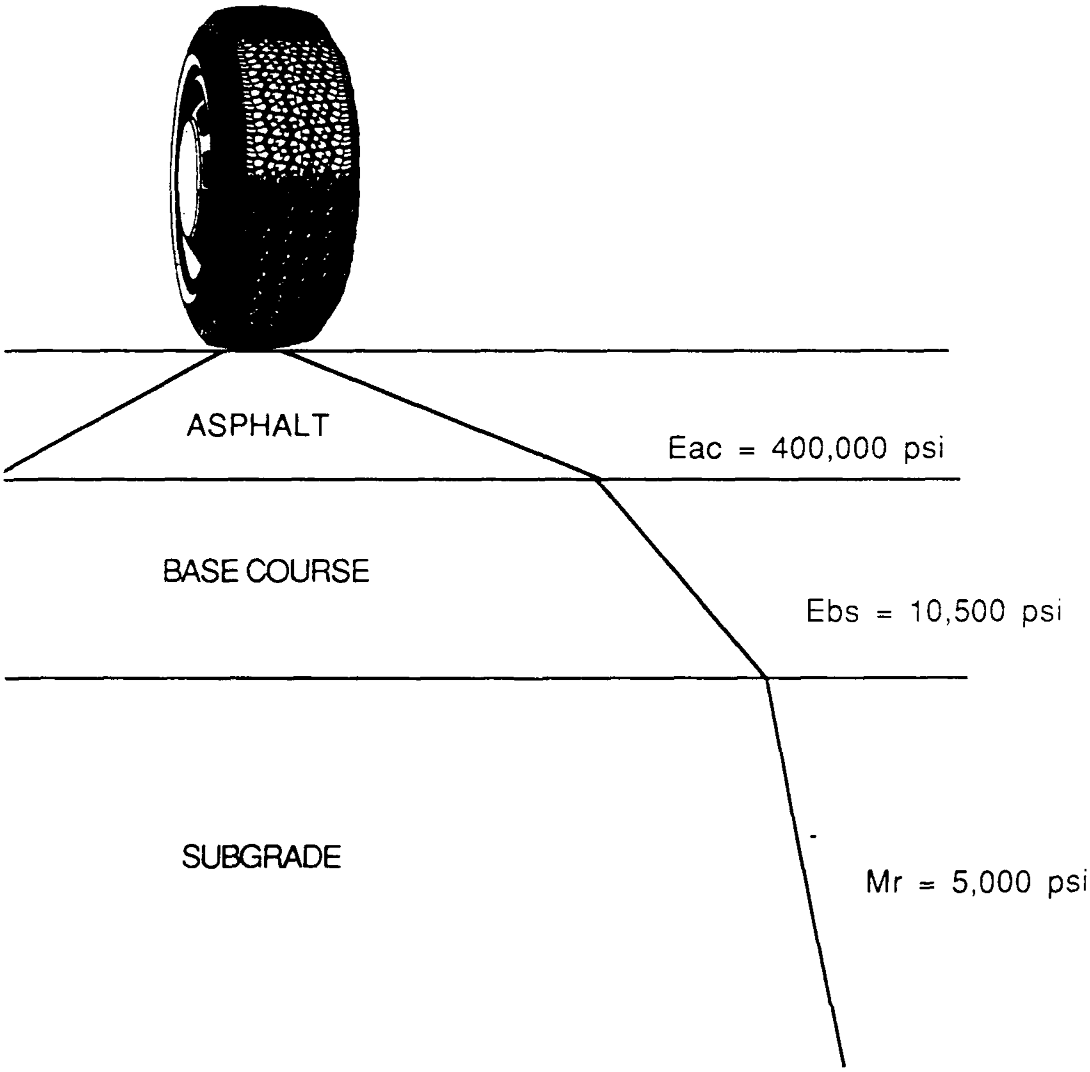


Figure 7 - Stress Distribution

The protection the upper asphalt and base coarse layers provide the subgrade is returned in the form of support. The subgrade (generally the least stiff layer) will also deflect when subjected to stresses. Since the stress from the load has been distributed over a much larger area by the upper layers, this stress is much lower than those in the material above. The lower the stresses and the stiffer the subgrade material, the less the subgrade and entire pavement structure will deflect. If the subgrade is a poor material , the resulting elastic deflections can cause fatigue cracking in the asphalt and any subgrade plastic deformation will result in ruts in the pavement surface. Stresses encountered at any level in the pavement structure can be theoretically determined using computer software packages based on elastic layer theory.

Environmental Effects. - "The seasonal variations of soil moduli are primarily induced by variations in soil moisture content, which depend on precipitation, temperature, soil gradation and permeability, surface distress level, and drainage conditions."[33] The AASHO Road Test results demonstrate the effects of moisture on the modulus.[34]

Moisture State	Equation
Dry	$8000\ \varnothing^{0.6}$
Damp	$4000\ \varnothing^{0.6}$
Wet	$3200\ \varnothing^{0.6}$

Following construction, the soil either stiffens as it dries to equilibrium or weakens as it approaches saturation.[35] This relationship is acknowledged by the AASHO Road Test (above) as well. But again, soil classification plays a role for if we know our soil type. we can make assumptions about its in situ moisture equilibrium, permeability, and drainage characteristics. Coupled with rainfall data, we can predict the moisture state (dry, damp, or wet) of the soil in various seasons. Western Washington experiences basically a wet mild winter season and a dry summer season. Subgrades are less variable with respect to seasonal variation than the base course which is closer to the surface. In a recent WA DOT

study, at the subgrade level, seasonal variations of unbound material moduli for Western Washington during the wet season only incurred a 10% reduction.[36]

Expected Resilient Moduli for Soils - Mahoney[37] determines roadbed soils can be broadly categorized by strength in the following manner:

CBR	Mr	Rating as a Subgrade
2	3,000	Poor
5	7,500	Fair
10	15,000	Good

Using the AASHTO CBR scaling factors and the USCS table for typical CBR ranges, a determination of expected ranges for resilient values can be calculated.

Soil Type	USCS Rating	CBR Range	Conv. Factor[38]	Conv. Mr	Median	Stiffness Criteria Rating
GW	Excellent	40-80	250	10,000-20,000	15,000	Fair-Excell
GP	Good-Excell	30-60	250	7,500-15,000	11,250	Fair-Good
GM	Good-Excell	20-60	250	5,000-15,000	10,000	Fair-Good
GC	Good	20-40	250	5,000-10,000	7,500	Fair
SW	Good	20-40	250	5,000-10,000	7,500	Fair
SP	Fair-Good	10-40	250	2,500-10,000	6,250	Poor-Fair
SM	Fair-Good	10-40	250	2,500-10,000	6,250	Poor-Fair
SC	Poor-Fair	5-20	1500/250	2,500-15,000	8,750	Poor-Good
ML	Poor-Fair	0-15	1500	0-15,000	7,500	Poor-Good
CL	Poor-Fair	0-15	1500	0-15,000	7,500	Poor-Good
OL	Poor	0-5	1500	0-7,500	3,750	Poor-Fair
MH	Poor	0-10	1500	0-15,000	7,500	Poor-Good
CH	Poor-Fair	0-15	1500	0-15,000	7,500	Poor-Good
OH	Poor	0-5	1500	0-7,500	3,750	Poor-Fair

Table 3 - TYPICAL CBR RANGES[39] & EXPECTED RESILIENT MODULUS

Since Mahoney's strength ratings and USCS subgrade ratings are somewhat consistent, the guidelines of 3,000, 7,500, and 15,000 psi equating to subgrades of "poor", "fair", and "good", respectively, based on stiffness characteristics alone can be used. Ratings of "excellent" will be applied to M_r values of much greater than 15,000 psi. This established criteria will be referred to as the "**stiffness criteria rating**".

CDF Ingredients.

Water - The functions of the water in the CDF mix are 1) to bring about the cement hydration process plus 2) to ensure workability and consolidation. Since CDF is not a concrete or a soil, water-cement ratios or compacted moisture contents don't necessarily apply. Like concrete however, a surplus of water will produce a weaker cemented mass causing some owners to write specifications with a water-cement ratio maximum. A higher water content improves flowability but decreases cohesiveness. Any excess "bleedwater" tries to "float" in the denser mix to the surface. Almost any natural water that does not have excessive impurities in it can be used as mixing water. Generally, drinking water sources are preferred. ASTM C94 provides guidance on acceptable water supplies for concrete which can be extended to include CDF.

Aggregates[40] - The quality of the aggregate, the aggregate grading, and the proportion of fine to coarse, all have an effect on durability. Since aggregates make up the bulk of the mix, it follows that they play an important role in the performance of a strong durable product. As in concrete, aggregates should "be clean, hard, strong, durable particles free of absorbed chemicals, coatings of clay, and other fine materials in amounts that could affect hydration and bond of the cement paste."[41] Aggregate is defined as either coarse (> #4 sieve) or fine (< #4 sieve).

The aggregate absorption properties and stockpile **moisture contents** are important in the overall mix design. Since any water additions to the mix are assumed to mix with the cement to form a paste (aggregate in saturated surface dry condition), a dry aggregate which absorbs water from the paste will greatly influence workability and strength of the mix. Conversely, stockpiles of saturated aggregate may have excess water which effectively raises the amount of water in the mix. Therefore, mix water must be adjusted to reflect the actual moisture content of aggregate when it varies from saturated surface dry.

The role of the **shape and texture** of aggregates in CDF is not quite clear. It plays an important role in both concrete and granular material performance. With concrete,

smooth, round particles require less paste to coat leaving more paste for workability. Crushed angular, rough particles have a larger surface area needing more paste and therefore, more water (higher w/c ratio) to achieve equivalent slump and workability. Angular particles also have more of a tendency to trap bleedwater in the transition zone, negating any cohesion advantages from the rough surface texture. The final reason rounded gravels may be preferred to crushed rock is due to the localized stress concentrations that may develop at the tip of the angular rock. The round particle will provide a broader more uniform load distribution at high stresses. On the other hand, for granular base course material, the rounded surfaces of gravel tend to shear under a load more easily . Crushed angular particles give the base course its strength through resistance to shear. Due to its relatively small cement contents, CDF's aggregate probably interacts more like a granular fill (or asphalt concrete) than a portland cement concrete. At higher cement contents (> 50 lbs/CY) the rounded particles may provide a higher compressive strength. In either case, however, the angular particles may counter the most prominent benefit of CDF, its flowability.

The maximum size of an aggregate is defined as the smallest initial sieve that 95% or more of the material passes. If CDF behaves more like a fill than a PCC, the **gradation** of the aggregate determines its structural capacity, and its vulnerability to frost heave and drainage problems. A well-graded aggregate base has a good particle size distribution which packs densely when compacted providing a firm (strong) surface. When particle sizes are relatively the same size (uniform) or the base composition is missing a specific range of particle sizes (gap-graded), then voids will exist after compaction which leads to a weak, shifting surface.

In areas of the country where ground water exists and the ground may freeze, **frost heave** (swell) can occur in soils with a high percentage of fine particles (passing the #200 sieve). This swelling combined with the subsequent loss of bearing capacity during the spring thaw, can cause serious damage to a pavement structure. As with moisture, frost

susceptibility was not studied in depth, although we know that fine-grained subgrades (silts & clays) will be more likely to wreak havoc on pavements in colder climates. In general, soils with $P_{200} > 3\%$ are at risk of frost heave.[42,43] We would therefore be somewhat concerned with CDF above the frost line since fly ash particles are so very small. Reports on CDF's susceptibility to freeze-thaw action conflict. Because of its makeup, it seems reasonable to assume that it would not perform well in freeze-thaw testing. In a HUD study, a mix with 150 lbs of cement, 200 lbs of fly ash, 2,590 lbs of fine aggregate, and 497 lbs of water was tested using ASTM test D 560-57, Freezing and Thawing Tests of Compacted Soil Cement Mixtures. The mix did not contain any air-entraining agents and the result was "that lean mix backfill does not have freeze-thaw resistance in the strength ranges tested."[44] In a large part of the country, including Seattle, the importance of freeze-thaw resistance is reduced since the frost line may not extend to the subgrade level.

Gradation also directly effects the permeability of the material layer and its ability to drain the excess water without losing its fine particle composition. Proper **drainage** of the subgrade is necessary to prevent pore water pressure build-up and discontinuous air voids when the material reaches 85% saturation. Under the instantaneous dynamic loads encountered under a pavement structure, the pore water pressure can lead to loss of shear strength and a pavement failure. Permeability (K) is a measure of drainage characteristics. A calibration of permeability values for a pavement structure is as follows:

K =	≤ 1 ft/day	1 ft/day	100 ft/day	1000 ft/day
	very poor	poor	good	excellent

Table 4[45] - Permeability

Although a plus in some applications, a potential drawback for CDF as it applies in a pavement structure is that its permeability is $1X10^{-5}$ to $1X10^{-7}$ cm/sec (.028 to .00028 ft/day). This will prevent the pavement base course from properly draining through the CDF subgrade. If the pavement cut is transverse and proper drainage was initially considered when placing the base course, the consequences could be minimal.

The compacted **unit weight** is a measure of what kind of compacted density we could expect in the field under ideal conditions (assuming tested moisture content was optimal). It is also expected that the more well-graded aggregate material will be more responsive to compaction by arranging particles in the most ideal (dense) manner. Uniformly graded material will be unable to fill air voids between uniform size particles since fines aren't present. The well-graded material should have a higher unit weight as a result. These denser materials should exhibit stiffer properties since air voids have essentially been eliminated. On a smaller scale, the fly ash fines in CDF are to sand as sand is to gravel in a granular fill completely filling all voids except those of migrating water. This accounts for its dense, low permeable structure.

Cement - Most cement used today is called Portland Cement invented in 1824 in England. It is manufactured from limestone and other raw materials which are pulverized and heated to 2700 F in a kiln. The heating transforms the limestone into a clinker containing the following important chemical compounds[46]:

Compound		Reaction Rate	Early Strength	Long-term Strength	Heat
Tricalcium Silicate	C_3S	Medium	Good	Good	Medium
Dicalcium Silicate	C_2S	Slow	Poor	Excellent	Low
Tricalcium Aluminate	C_3A	Fast	Good	Medium	High
Tetracalcium Aluminoferrite	C_4AF	Medium	Good	Medium	Medium

The clinker is then ground extremely fine so that its surface area is large enough so that when combined with water it can react efficiently. There are five common types of Portland Cement used in concrete depending on its application. The difference in types is achieved by varying its active chemical compound proportions to desired results (see Table 5).

Type I	Normal	Used in most general purpose construction.
Type II	Moderate Sulfate	Lower heat of hydration and moderate sulfate resistance. Used in structures of moderate size where precaution should be taken against minor sulfate attack (drainage structures, retaining walls or abutments).
Type III	High Early Strength	Used when need to put into service as soon as possible or to raise heat of hydration in cold weather.
Type IV	Low Heat	Used in huge structures where excessive heat of hydration may be a problem.
Type V	Sulfate Resistant	Used when concrete is exposed to seawater or soil.

Compound Composition %

Cement	C_3S	C_2S	C_3A	C_4AF
Type I	55	19	10	7
Type II	51	24	6	11
Type III	56	19	10	7
Type IV	28	49	4	12
Type V	38	43	4	9

Table 5[47] - Portland Cement Chemical Compounds

Cement has a specific gravity of 3.15 (dense) and can be purchased in 94 lb sacks or in bulk.

Fly Ash[48] - When pulverized coal is burned in electric power generating plants, the mineral impurities fuse in the exhaust gas. The suspended material solidifies into tiny glass-like balls and is collected as fly ash by mechanical means at the plant. Pozzolans such as fly ash are often added to fresh concrete in structural applications when flowability and long term strength are important. When fly ash replaces a percentage of the cement in the mix design, its tiny spherical balls lubricate the mix producing a concrete that is easily placed at a reduced price. Fly ash is primarily a silicate glass with a typical particle size

under 20μm.[49] For this reason, the fly ash in CDF makes CDF what it is - a dense "flowable fill".

An added advantage is that, by definition, a pozzolan reacts with the cement hydration by-product compounds to add strength. "A pozzolan is a siliceous or aluminosiliceous material that in itself possesses little or no cementitious value but will, in finely divided form and in the presence of water, chemically react with the calcium hydroxide released by the hydration of portland cement to form compounds possessing cementitious properties."[50]

ASTM C-618 classifies fly ash into two general categories separate from other pozzolans such as clays, shales, volcanic ashes, and diatomaceous earths (Table 6).

Type	Description	Calcium Content	Carbon Content
Class F	Fly Ash with Pozzolanic Properties	< 10%	< 10%
Class C	Fly Ash w/ Pozzolanic & Cementitious Properties	10-30%	< 2%
Class N	Other Pozzolans		

Table 6[49] - Classes of Pozzolans

Class C fly ash has the advantage of containing higher CaO mineral content which can hydrate like cement in addition to its pozzolanic properties. Fly ash powder resembles cement in appearance. It's color is typically tan or grey and its specific gravity ranges between 2.2 and 2.8.

Air-Entraining Agent (AEA) may be introduced to help manage the potential freeze-thaw problem that has already been discussed. Its measure must be controlled because air contents mean voids and some loss in compressive strength.

CDF Mixes - In some respects similar to concrete, CDF component materials can be varied to match the intended application. Fly ash and water can be added to increase slump and flowability. Coarse aggregate and cement can be added to increase compressive

strength. Pozzolanic recommends the following range for each ingredient by weight (lbs) per cubic yard (SSD):

Fine Aggregate (+Coarse if used)	Cement	Fly Ash	Water
2,000 - 3,200	30 - 100	250 - 350	350 - 800

Obviously, the more cement that is added the more CDF behaves like a concrete, stiff and strong, but difficult to excavate and with possible shrinkage cracks. The city of Everett, Stoneway Concrete, and Associated Sand & Gravel have eliminated the coarse aggregate from their CDF mix for most applications. It was determined that the more expensive coarse aggregate created CDF that was stronger than necessary - making it sometimes difficult to excavate. The following summary was constructed to display the variety of mixes possible:

Project/Spec	Comp Strength	Course Agg	Fine Agg	Cement	Fly Ash	Water	Slump
[51]Delmarva Power Co. Cooling Tower Facility Saved $614,000 12,800 CY Pipe bed & Backfill	100 psi 200 psi 300 psi	- - -	- - -	5% 10% 15%	95% 90% 85%	- - -	- - -
[52]Unpublished 1981 U.S. HUD Spec Backfill, Pipe Bedding, Pavement Subbase, Foundation Stabilization	50-200 psi	if blend, max size = 1"	2,200 to 3,000 lbs/CY ASTM C-33	125 - 200 lbs/CY Type I or II	50 to 400 lbs/CY ASTM 618	350 to 800 lbs/CY	-
[53]Seattle - Metro Bus tunnel stations, 25,000 CY	80-100 psi	-	2450 lbs/CY	30 lbs/CY	300 lbs/CY	300 lbs/CY	-
Kenmore Sewer [54]Interceptor trial mixes 1985	435 psi 485 psi 585 psi	- - -	- - -	100 lbs 125 lbs 150 lbs	300 lbs 250 lbs 350 lbs	260 lbs 270 lbs 250 lbs	8.5" 9.5" 9.5"
[55]Fairmont, WV stabilize subsidence of mine 1983	200 psi			15,000 tons	125,000 tons		
[56]Ohio Ready Mixed Concrete Assoc. recommendation	-	blend	2,910 lbs	50 lbs	250 lbs	500 lbs max	
[57]Ohio State Route 7 Sewer Trench backfill	-	-	2,700 lbs	100 lbs	250 lbs	500 lbs	-

[58]Mt. Baker Ridge Tunnel (I-90) 784 CY 1984	-	1" minus	blend	50 lbs	300 lbs	-	8-10"
[59]Iowa Dept. of Transportation Spec. 1984	-	blend 100% < 3/4"	2,600 lbs 0-10%< #200	100 lbs	300 lbs	580 lbs (70 gal)	
[60]City of Everett, WA Spec Pavement Utility Repair	-	-	3200 lbs Everett #2	50 lbs I or II	250 lbs Class F	for slump	as desired
[61]City of Selah, WA Spec for Pavement base 1984	-	2,000 lbs 3/4 course	1,450 lbs	75 lbs	400 lbs	167 lbs (20 gal)	3 - 4"
[62]Utah Dept. of Transportation, 1986	-	1,270 lbs ASTM C-33 67	1,870 lbs ASTM C-33	25 lbs Type I-II ASTMM C-150	400 lbs Class F ASTM C-618	250 lbs (30 gal)	-
[63]City of Salt Lake, UT field tests, 1982	-	-	2,700 lbs	50 lbs Type I	498 lbs Type F	383 lbs	9.5"

Table 7 - Possible CDF Mixes

Curing[64] - After mixing and placing, the CDF should be adequately cured to prevent the loss of moisture and to control the temperature in the same manner as concrete. Both parameters are essential for hydration and strength gain. It is desirable to keep a uniform temperature throughout the concrete or CDF mass and to protect the structure from early loads, impact, or vibration during the initial curing period. The cement needs a high humidity environment to continue hydration, so the potential for evaporation must be eliminated as much as possible.

TESTING PROGRAM

Materials.

Soils - Results of resilient modulus testing on CDF would be of no value without being able to compare the results with those of common soil materials. Since current practice recommends the use of resilient modulus tests to determine soil stiffness and resultant suitability as a pavement subgrade, it seems it would be most helpful to the civil engineer to have an expected range of resilient modulus values for various soil groups with which he could make preliminary assessments of existing conditions. After conducting extensive

literary searches to use as a comparison for CDF, it was determined that such published information is sparse.

To compare CDF resilient values with those of typical subgrades, it was then appropriate to look at what correlation could be made between soil groups and actual resilient modulus values obtained in laboratory testing. It was presumed that tested stiffnesses would substantiate published AASHTO and USCS suitability generalizations. A literature review and additional laboratory tests were conducted as part of the investigation. In addition, a study conducted by the Washington State Transportation Center (TRAC - a joint venture between the two state universities and the Washington State Department of Transportation.) provided local data. Using the above sources, it was possible to obtain a fairly reasonable data base of resilient moduli covering most soil index groups.

Subgrade Stresses - The typical pavement structure used for representative stress calculations was a 6" asphalt concrete (E_{AC} = 400,000) & 6" granular base (conservative E_{bs} = 10,500). Since stresses are a function of the subgrade resilient modulus, M_r (subgrade stiffness) was initially approximated at 10,000 psi. Using an elastic layer computer program (ELSYM5) and simulating a 9,000 lb wheel load (Equivalent Single-Axle Load), stresses representative of an 18,000 lb equivalent axle were computed to be:[65]

deviator stress = 7.01 psi

confining pressure = 1.53 psi

bulk stress = 11.6 psi

Even though overburden confining pressures increase at increasing depth, we would rarely expect to see Ø or σ_d greater than 12 psi in the subgrade since the stresses from applied loads dissipate much more rapidly at increasing depths. At the low confining and applied pressures experienced in the subgrade, it is possible that low stiffnesses may be experienced for "excellent" rated granular materials since they show much more positive sensitivity to stresses. An attempt was made to determine regression equations for all

laboratory tested soil and CDF samples so that modulus values could be compared at equivalent stress levels.

CDF Mix Design. - The 4" X 8" CDF cylinders were prepared at Stoneway Concrete in Renton with representatives from Pozzolanic (Dennis Augustine and Jenny Flechsig). The following materials were used in laboratory test batches:

Material	Type	Source	Specific Gravity	ASTM Specification
Cement	II	Ash Grove, Montana City, MT	3.15	ASTM C-150
Fly Ash	F	Pozzolanic, Centralia, WA	2.20	ASTM C-618
Sand	Building	Glacier Pit, Steilacom, WA	2.67	
Sand	Concrete	Glacier Pit, Steilacom,WA	2.63	ASTM C-33 Fine
Water	Tap	Renton, WA	1.00	-

Table 8 - Tested CDF Composition

The following mix combinations were prepared and tested:

Mix	Fine Aggregate (+Coarse if used)	Cement	Fly Ash	Water
CDF 351	2,450	30	300	300
Typical (451)	2,585	40	300	268
No Fly Ash	2,690	40	0	483

All mixes contained 10 oz/CY of an air entraining agent (DARAVAIR).

Stoneway's most popular seller CDF "351" is a mix using only 30 lbs of cement, 300 lbs of fly ash, and combined with a coarse sand filler. The two CDF "351" cylinders were samples previously taken from a fill project and supplied to the University. Their resulting resilient moduli were used for comparison. Due to the material's application as a pavement subgrade, a stiffer mix, using 40 lbs of cement (CDF 451), was prepared and the majority of testing was conducted on these samples. A sand meeting ASTM C-33 Fine

specifications was used for aggregate since in the small 4" diameter cylinders, large aggregate particles might have distorted the results. One cylinder containing no fly ash was tested to determine its effect on the overall stiffness. Building sand was used in the CDF 351 mix. The concrete sand used in the mix with the higher cement content was a more uniformly graded sand meeting ASTM C-33 specifications. The sands gradation curves are as follows:

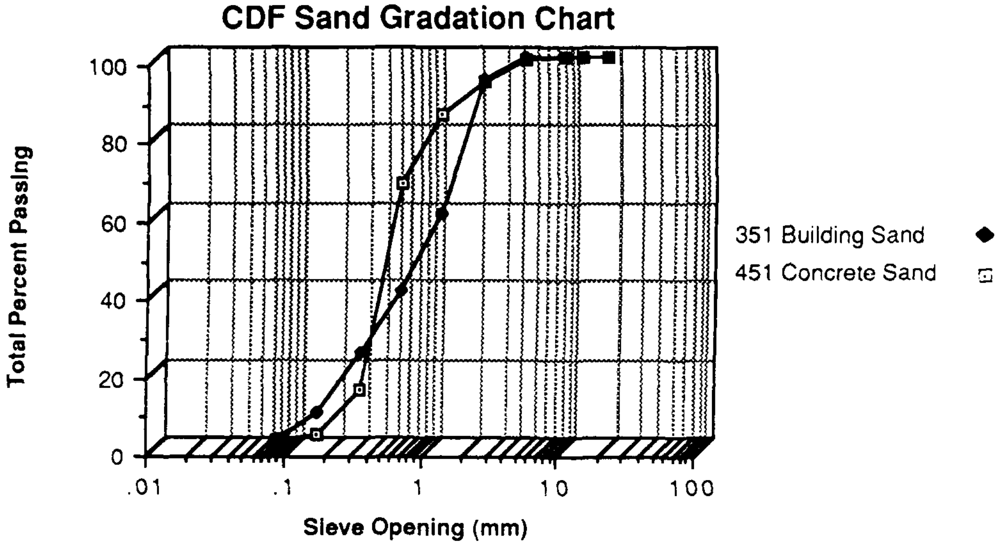


Figure 8 - Sieve Analysis of CDF Sands

Procedures

ASTM procedures for the preparation of fresh concrete specimens was undertaken for the CDF. Unit weight and yield calculations were made following ASTM C 138 procedures with the exception that no rodding of the mix was conducted and weights were taken in the cylinder molds. Due to its flowable nature, slump is virtually immeasurable by ASTM C 143 procedures. Instead equivalent slumps were determined by measuring the horizontal diameter of the circle formed when the inverted slump cone is lifted from the smooth surface (mortar cone test). Specimens were prepared in 4" diameter by 8" high plastic cylinder molds and were not rodded. Initial specimens were not capped prior to testing. Instead porous stone caps were placed at either end of the cylinder. After the first

4 cylinders were tested, a hydrostone cap was placed on subsequent cylinders to help provide a more uniform loading surface since the surface of the hardened mix is somewhat frangible. Obviously, CDF specimens were not compacted as required for typical subgrade soils. All samples were cured at least thirty days prior to resilient modulus testing.

Resilient modulus testing was conducted using AASHTO T274 - Resilient Modulus of Subgrade Soils procedures with some modifications. Since the resilient modulus apparatus at the University of Washington is set up to measure load and deflection on the exterior of the triaxial cell, testing was conducted in this configuration with no attempt to modify it for cylinders that exceeded M_r values of 15,000 psi. The repeated loads were applied pneumatically with a frequency of about 0.5 Hz. Cylinders were "conditioned" and tested at deviator and confining pressures described for granular soils in AASHTO T274 except that deviator stresses larger than 10 to 12 psi were not conducted since subgrades will generally not encounter loads of this magnitude. Also, preliminary tests at zero confining and higher deviator stresses failed due to excessive straining. Using the AASHTO designated deviator loads and confining pressures, K_1 and K_2 values can be determined for each sample's bulk or deviator stress regression equation. The stresses calculated for a "typical" pavement structure at the subgrade level (Figure 10) can then be inserted in the regression equation to compare stiffnesses of different materials or samples under the same stresses.

Because early tests showed signs of breaking down after a high repetition of loads at higher stresses, it seemed necessary to try to evaluate CDF's susceptibility to **fatigue**. Plastic deformations were measured for every load sequence which were cumulatively added to represent the amount of settlement.

An approximate number of equivalent axle loads can be calculated to equate to an equivalent number of years of service life the cylinder was exposed to. To conduct a traffic analysis it is necessary to consider traffic volume, composition, and axle weights with the goal being to develop the equivalent number of 18,000 lb equivalent single axle loads.[66]

By using AASHTO Design of Pavement Structures (Appendix D) and assuming a structural number (SN) of 4 and terminal serviceability index of 2.5, equivalency factors can be interpolated to represent the damage done to the pavement under varying loads.

Lab tests for soils were mostly conducted at optimal moisture content (optimal for compaction to achieve maximum density) or in situ moisture content. CDF cylinders were cured and stored in a fog room until testing. Moisture content was determined at the conclusion of the resilient modulus testing.

RESULTS

Data & Calculations.

Fresh CDF. - The Absolute Volume Method for determining concrete mix proportions was used for batching the CDF. The following is an example of the calculations made for the typical mix:

Batch Size = .03 yd^3 Aggregate Moisture Content = .06 above SSD

Material	SSD Weights	SG	Solid Volume	Adj. Weights	Batch Weights	ft^3 Equivalent
Cement	40	3.15	.20		1.2	1.48
Fly Ash	300	2.25	2.14		9.0	11.11
Sand	2585	2.63	15.75	2740	82.2	101.48
Water	255	1.0	4.09	100	3.0	3.70
Ent Air			4.86			
TOTALS	3180		27.04		95.4	117.77

Volume of Cylinder = $\pi r^2 * h = \pi (2/12)^2 * (8/12) = .0582\ ft^3$

Weight of Cylinder Mold = .31 lbs

Weight of Mold & Mix = 7.17 lbs

Unit Weight of Fresh Mix = (7.17-.31) / (.058) = 118.27 lbs/ft^3 ~ 117.77

Yield = 3180 / 118.27 = 26.89 ft^3

Slump as traditionally measured > 11"

Slump measured as diameter of circle formed on flat surface (mortar cone) = 21.5"

Cylinder Specimen Data. - Data for each specimen tested was recorded on a spreadsheet similar to that recommended in AASHTO T274 except that not all the data was needed for our purposes. An example of the specimen data worksheet is as shown in Table 9.

Calibration. - The resilient modulus apparatus was calibrated between each test cylinder using a 5 kip proving ring for load and a vernier micrometer for calibrating deflection readings. An example of the measurements for calibrating the external loading device and the externally mounted linear variable differential transformers (LVDT) is included in Figure 9.

Subgrade Stresses - The ELSYM5 stress computations for an 18 kip single axle load are shown in Figure 10.

Plastic Strain. - The permanent deformation was measured from the time the initial load was applied. Measurements therefore include the initial seating of the apparatus and were then larger than they should be. Cylinders also showed signs of long term rebound (after testing at high stresses, plastic strain was negative while loading at lower stresses). This also caused the plastic strain summation to be overly conservative since there was no measure of rebound when the cylinder rested overnight. Only sample 1C was subjected to at least 3000 loading cycles at each loading combination with recordings taken at 200 and 3000 cycles each. The plastic strain spreadsheet shown in Table 10 was used to compute the Equivalent Single Axle Loads (ESALs) for sample 1C and to compute the total strain on the cylinder.

Resilient Modulus - After repeating the loading the prescribed number of times (at least 200), about ten readings were taken, measured, and averaged. An example of a test recording and measurements is displayed in Figure 11. Data was taken for each cylinder at every combination of confining and deviator loads on a worksheet like that in Table 11. The averaged measurements were compiled on the spreadsheet. This spreadsheet was used

CYLINDER SPECIMEN DATA WORKSHEET

RESILIENT MODULUS

AASHTO T274

Date 7/12/90

Compaction Method ____________

Soil Sample		CDF
Mix		451
Sample #		1G
Specific Gravity		____________
Specimen Measuremen		
	Top	4.00
Diameter	Middle	4.00
	Bottom	4.00
	Average	4.00
Membrane Thickness		0
Net Diameter		4.00
Ht Spec.+Cap+Base		____________
Ht Cap+Base		____________
Length (in)		7.875

Soil Specimen Wt	
Initial Wt of Container+Wet Soil (gms)	3733
Final Wt of Container+Wet Soil (gms)	709
Wt Wet Soil Used (gms)	3024
Soil Specimen Volume	
Area (in2)	12.56
Volume (in3)	98.96
Wet Density (pcf)	116.30
Water Content (Tested)	
Wt of Pan (gms)	683.0
Wt of Pan+Sample (gms)	3663.0
Wt of Pan+Dried Sample (gms	3371.0
Compaction Water Content, w	10.86%
Volume of Solids, Vs (in3)	#VALUE!
Volume of Water, Vw (in3)	18.07
Volume of Voids, Vv (in3)	#VALUE!
% Saturation	#VALUE!
Dry Density (pcf)	104.91

TABLE 9

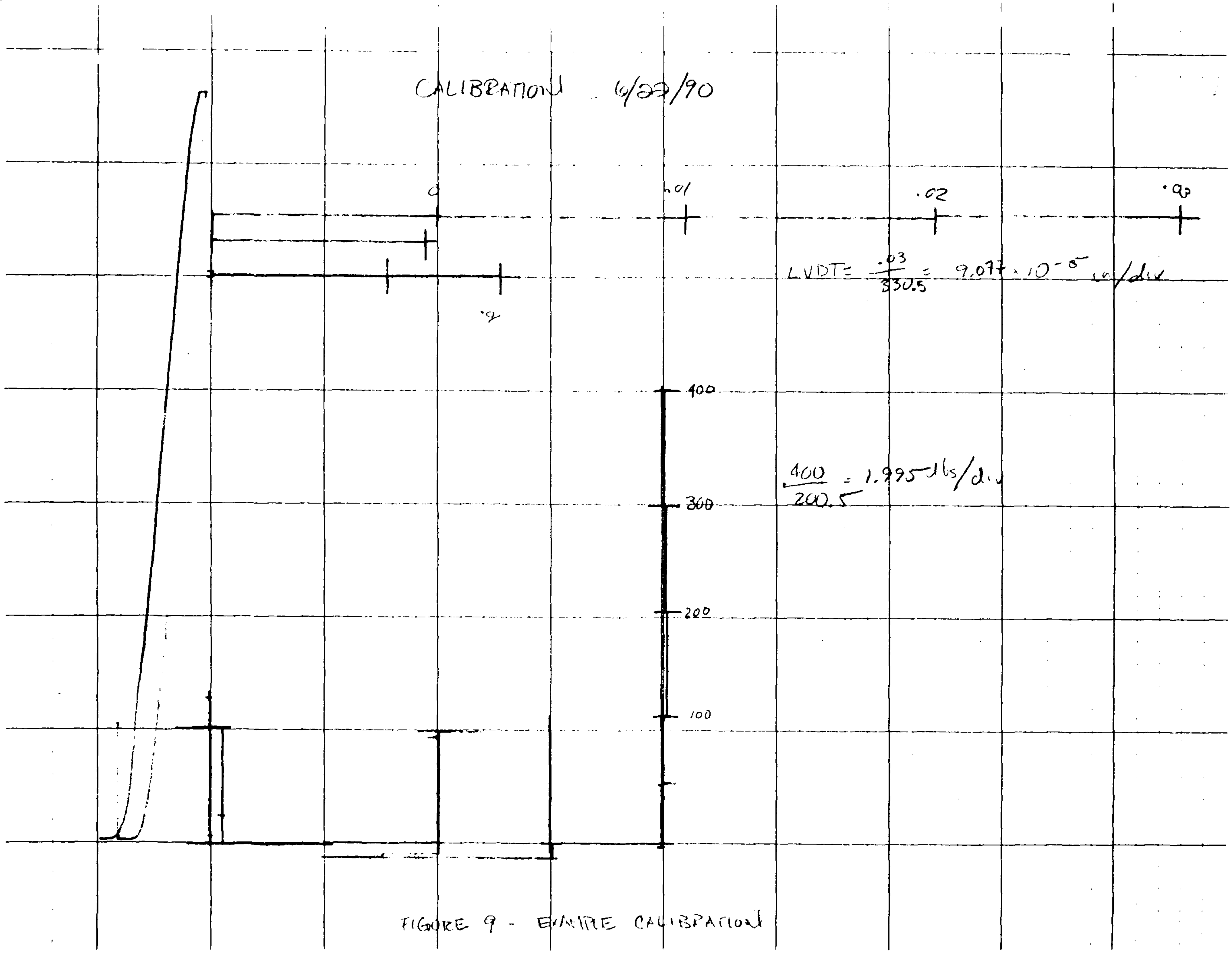


FIGURE 9 - EXAMPLE CALIBRATION

ELSYM5 ELASTIC LAYER ANALYSIS
& STRESS CALCULATIONS

ELASTIC SYSTEM - 6" AC <u>OVERBURDEN</u>

LAYER	ELASTIC MODULUS	POISSONS RATIO	THICKNESS
1	400000.	.350	6.000 IN
2	10500.	.400	6.000 IN ← CONSERVATIVE E_{bs}
3	10000. ← Assume	.400	SEMI-INFINITE

ONE LOAD(S), EACH LOAD AS FOLLOWS

TOTAL LOAD..... 45113.00 LBS
LOAD STRESS.... [redacted]
LOAD RADIUS.... 120.00 IN

OVERBURDEN
LOAD CALCULATION

Use a 20' diameter circular load to simulate AC & Base overburden weight.

LOAD	LOCATED AT X	Y
1	.000	.000

RESULTS REQUESTED FOR SYSTEM LOCATION(S)

DEPTH(S)
Z= 12.10
(X,Y) POINT(S)
.00 .00

$$\text{Overburden Area} = \frac{\pi (20')^2}{4} = 314 \text{ ft}^2$$

$$\text{AC Load} = \frac{6''}{12''} \times \frac{145 \text{ lbs}}{\text{ft}^3} \times 314 \text{ ft}^2 = 22,765 \text{ lbs}$$

$$\text{Base Course} = \frac{6''}{12''} \times \frac{142.3 \text{ lbs}}{\text{ft}^3} \times 314 \text{ ft}^2 = \underline{22,348 \text{ lbs}}$$

$$\boxed{45,113 \text{ lbs}}$$

Z= 12.10 LAYER NO. 3

X Y
.00 .00

NORMAL STRESSES
SXX -.6146E+00 = σ_3'
SYY -.6146E+00 = σ_2'
SZZ -.7894E+00 = σ_1'

SHEAR STRESSES
SXY .0000E+00
SXZ .0000E+00
SYZ .0000E+00

PRINCIPAL STRESSES
PS 1 -.6146E+00
PS 2 -.6146E+00
PS 3 -.7894E+00

PRINCIPAL SHEAR STRESSES
PSS 1 .1874E+00
PSS 2 .0000E+00
PSS 3 .1874E+00

FIGURE 10

DISPLACEMENTS

ELASTIC SYSTEM – [redacted]

LAYER	ELASTIC MODULUS	POISSONS RATIO	THICKNESS
1	400000.	.350	6.000 IN
2	10500.	.400	6.000 IN
3	10000. (ASSUME)	.400	SEMI-INFINITE

ONE LOAD(S), EACH LOAD AS FOLLOWS

TOTAL LOAD..... 9000.00 LBS
LOAD STRESS.... 100.00 PSI
LOAD RADIUS.... 5.35 IN

LOAD	LOCATED AT X	Y
1	.000	.000

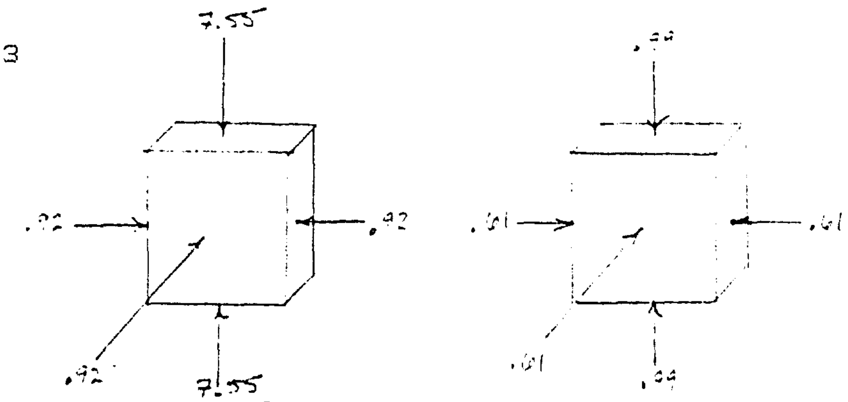
RESULTS REQUESTED FOR SYSTEM LOCATION(S)

DEPTH(S)
Z= 12.10
X-Y POINT(S)

X	Y
.00	.00

Z= 12.10 LAYER NO. 3

X	Y
.00	.00



TIRELOAD

NORMAL STRESSES
SXX -.9152E+00
SYY -.9152E+00
SZZ -.7550E+01

SHEAR STRESSES
SXY .0000E+00
SXZ .0000E+00
SYZ .0000E+00

PRINCIPAL STRESSES
PS 1 -.9152E+00
PS 2 -.9152E+00
PS 3 -.7550E+01

PRINCIPAL SHEAR STRESSES
PSS 1 .3318E+01
PSS 2 .0000E+00
PSS 3 .3318E+01

$\sigma_1 = 7.55 + .99 = 8.54$ psi
$\sigma_2 = .92 + .61 = 1.53$ psi
$\sigma_3 = .92 + .61 = 1.53$ psi
BULK STRESS = $\boxed{\theta = 11.6 \text{ psi}}$
DEVIATOR STRESS $\sigma_d = \sigma_1 - \sigma_3 =$
$8.54 - 1.53 = \boxed{7.01 \text{ psi}}$

DISPLACEMENTS
UX .0000E+00
UY .0000E+00

FIGURE 10 (cont)

PLASTIC STRAIN / ESAL WORKSHEET

Sample # 1C Length = 7.88

LVDT = 2.00E-04 9.08E-05

Deviator (psi)	Confining (psi)	Reps	Axle Load _/7.01*18 (kips)#	Equiv. Factor App. D*	ESALS	Δ (div)	Plastic Δ (in)	Cum Δ (in)
5.00	5	200	12.84	0.287	57.40	39	0.0078	0.0078
10.00	5	200	25.68	3.747	749.40	48	0.0096	0.0174
10.00	10	200	25.68	3.747	749.40	10.5	0.0021	0.0195
9.74	10	607	25.01	3.405	2066.84	3	0.0006	0.0201
1.67	10	221	4.29	0.004	0.88	0	0	0.0201
1.47	10	1016	3.78	0.003	3.05	0	0	0.0201
2.71	10	3854	6.95	0.026	100.20	-7	-0.0014	0.0187
5.49	10	248	14.10	0.401	99.45	0	0	0.0187
5.51	10	1071	14.15	0.407	435.90	0	0	0.0187
5.46	10	3236	14.01	0.389	1258.80	0	0	0.0187
8.61	10	220	22.12	2.138	470.36	0	0	0.0187
6.00	10	1015	15.40	0.568	576.52	10	0.002	0.0207
9.22	10	3215	23.67	2.758	8866.97	3.5	0.0007	0.0214
13.94	10	213	35.80	14.09	3001.17	0	0	0.0214
13.90	10	1234	35.69	13.92	17177.28	0	0	0.0214
13.87	10	3428	35.60	13.78	47237.84	4	0.0008	0.0222
17.88	10	242	45.91	41.067	9938.21	0	0	0.0222
17.90	10	1240	45.95	41.215	51106.60	2	0.0004	0.0226
17.88	10	4395	45.91	41.067	180489.47	4	0.0008	0.0234
1.51	5	217	3.88	0.003	0.65	0	0	0.0234
2.27	5	1196	5.84	0.012	14.35	2.5	0.0005	0.0239
2.12	5	3293	5.45	0.01	32.93	3	0.0006	0.0245
4.39	5	214	11.28	0.173	37.02	0	0	0.0245
4.24	5	3494	10.89	0.151	527.59	0	0	0.0245
8.79	5	207	22.57	2.318	479.83	2	0.0004	0.0249
8.70	5	2195	22.33	2.222	4877.29	3.5	0.0007	0.0256
8.76	5	4672	22.50	2.29	10698.88	2	0.0004	0.026
13.34	5	215	34.24	11.672	2509.48	1	0.0002	0.0262
13.38	5	2013	34.36	11.858	23870.15	5	0.001	0.0272
13.41	5	6852	34.44	11.982	82100.66	3.5	0.0007	0.0279
17.58	5	208	45.14	38.218	7949.34	0	0	0.0279
17.59	5	3650	45.17	38.329	139900.85	3.5	0.0007	0.0286
2.42	1	210	6.23	0.016	3.36	3	0.0006	0.0292
2.17	1	2660	5.56	0.011	29.26	2.5	0.0005	0.0297
4.55	1	213	11.67	0.195	41.54	0	0	0.0297
4.48	1	2991	11.49	0.185	553.34	0	0	0.0297
5.08	1	276	13.04	0.304	83.90	0	0	0.0297
5.08	1	2895	13.04	0.304	880.08	2	0.000182	0.02988
8.03	1	231	20.62	1.662	383.92	0	0	0.02988
8.03	1	3180	20.63	1.665	5294.70	0	0	0.02988
10.19	1	200	26.17	4.021	804.20	0	0	0.02988
10.07	1	1553	25.86	3.839	5961.97	0	0	0.02988
10.25	10	214	26.32	4.118	881.25	0	0	0.02988
				Total ESALs =	612,302.3		Total Δ	0.02988
							Total Strain	0.00379
							% =	0.38%

* AASHTO p. D-6 (Single Axle, SN = 4, Pt = 2.5)

TABLE 10

to calculate the stresses, strains, and resulting resilient moduli at each stress combination for each cylinder tested. The spreadsheet calculated K_1 and K_2 values for a bulk stress regression equation as well. Worksheets for all CDF samples are available in Appendix B.

Analysis of Results.

Plastic Strain. - From the worksheet, the total equivalent 18k loads applied to the cylinder was approximately 612,300 (Table 10). For comparison, an analysis conducted on Stevens Way on the campus of the University of Washington this fall determined that its mean annual traffic equalled 83,700 ESALs.[67] Therefore, the laboratory simulation equated to about 7.3 years of traffic if the CDF had been placed as a subgrade for a utility cut across Steven's Way. For additional comparison, the Asphalt Institute estimates annual traffic volume by type of street or highway as follows:

Type of Street or Highway	Traffic Class	Estimated 18 KEAL
1. Parking Lots 2. Light traffic residential streets and farm roads.	I	5,000
1. Residential streets 2. Rural farm and residential roads	II	10,000
1. Urban and rural minor collectors	III	100,000
1. Urban minor arterial and light industrial streets. 2. Rural major collector and minor arterial highways	IV	1,000,000
1. Urban freeways and other principal arterial highways. 2. Rural interstate and other principal arterial highways.	V	3,000,000
1. Urban interstate highways 2. Some industrial roads.	VI	10,000,000

Table 12 - Asphalt Institute Traffic Classifications[68]

The deformation measured was the total plastic deformation at the time of the reading. It was observed that when a lighter applied load followed a heavier load, there was

RESILIENT MODULUS SAMPLE 1C

5/6/90 $\sigma_3 = 10$ psi $\sigma_d = 3$ psi

200 loads

#	y	x
1	18.5	6
2	17.5	6.5
3	18	6
4	18.5	5.5
5	18.5	6.5
6	18	5.5
7	18	6.5
8	18	6.5
9	18	6
10	18	6
11	18	6
12	18	5.5
13	18.5	6
AVG	18.12	6.04

248

ϕ

FIGURE 11 - EXAMPLE TEST READINGS MEASUREMENTS

SAMPLE RESILIENT MODULUS WORKSHEET

RESILIENT MODULUS

AASHTO T274

Start 7/12/90

Finish 7/13/90

Measurements		Constants		Specimen	
Length	7.88	LVDT	9.33E-05	Type	DF Normal
Area	12.57	Load Cell	1.9139	#	1G

Chamber Pressure ∂3 (psi)	Nominal Dev. Stress ∂d (psi)	Load Cell Reading (squares)	Deviator Load Ld (lbs)	Deviator Stress ∂d (psi)	Recoverable Deformation Reading (squares)	Recoverable Deformation Δ (inches)	Strain E (in/in)	Bulk Stress ø (psi)	Resilient Modulus Mr (psi)
10.00	1.5	11.10	21.24	1.69	3.15	2.94E-04	3.73E-05	31.69	45,293
10.00	3.0	20.50	39.23	3.12	5.05	4.71E-04	5.98E-05	33.12	52,177
10.00	5.0	32.90	62.97	5.01	8.60	8.02E-04	1.02E-04	35.01	49,172
10.00	8.0	47.60	91.10	7.25	10.50	9.80E-04	1.24E-04	37.25	58,269
10.00	10.0	58.40	111.77	8.89	13.85	1.29E-03	1.64E-04	38.89	54,198
5.00	1.5	11.40	21.82	1.74	3.35	3.13E-04	3.97E-05	16.74	43,740
5.00	3.0	21.30	40.77	3.24	5.50	5.13E-04	6.52E-05	18.24	49,778
5.00	5.0	30.90	59.14	4.71	6.65	6.21E-04	7.88E-05	19.71	59,725
5.00	8.0	49.50	94.74	7.54	12.95	1.21E-03	1.53E-04	22.54	49,131
5.00	10.0	58.05	111.10	8.84	13.70	1.28E-03	1.62E-04	23.84	54,463
1.00	1.5	11.00	21.05	1.68	3.70	3.45E-04	4.38E-05	4.68	38,213
1.00	3.0	20.85	39.90	3.18	5.10	4.76E-04	6.04E-05	6.18	52,548
1.00	5.0	31.55	60.38	4.81	7.90	7.37E-04	9.36E-05	7.81	51,332
1.00	8.0	49.20	94.16	7.49	10.45	9.75E-04	1.24E-04	10.49	60,515
1.00	10.0	60.05	114.93	9.15	9.50	8.86E-04	1.13E-04	12.15	81,247
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!
			0.00	0.00		0.00E+00	0.00E+00	0.00	#DIV/0!

K1= 48,401 K2= 0.029

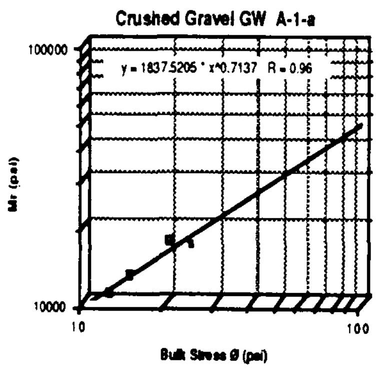
TABLE 11

noticeable long-term rebound in the sample. Unfortunately, any rebound that occurred overnight was not measured, probably causing values to be much higher than in actuality since the same deformation was measured cumulatively. Regardless, the combined plastic strain totalled .00379 in/in after the 7.3 years of equivalent service life. Considering these are extremely conservative figures, the CDF appears to hold up very well under repeated loads.

Resilient Modulus of Soils

Sands & Gravels

For comparison purposes and to broaden the data base, five soils were tested in the laboratory to determine their suitability as a subgrade or base course. Those tested primarily for their properties as a base course consisted of a well-graded crushed rock (GW), a sand and gravel blend (GM), and a uniformly graded crushed rock scalped on the 1/2" sieve (GP). The two less coarse soils were a poorly graded sand (SP) and a sandy clay (SC). All were compacted at optimum moisture content using the standard proctor method. The obtained results are shown in Table 13 and Figures 12 to 16.



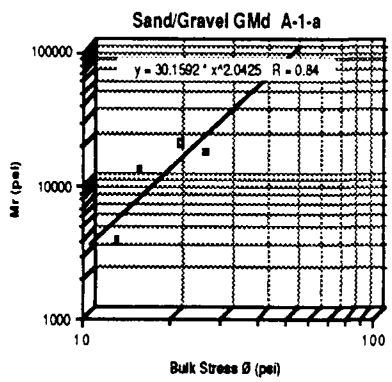


Figure 12 - Crushed Gravel Results **Figure 13** - Sand-Gravel Blend Results

COMPARISON OF LABORATORY RESILIENT MODULI TO SOIL INDEX

Soil Type	AASHTO	Index Rating	USCS	Index Rating	Regression Equation	R^2	# Data Pts	Density (lb/ft3) Wet	Density (lb/ft3) Dry	Modulus @ Ø=11.6 psi	Stiffness Criteria Rating
Crushed Rock	A-1-a	Excellent	GW	Excellent	$1{,}838Ø^{0.714}$	.92	4	143.7	134.5	10,577	Fair-Good
Sand / Gravel	A-1-a	Excellent	GMd	Good-Excellent	$30.16Ø^{2.043}$	.71	4	142.2	132.6	4,509	Poor
Scalped Rock	A-1-a	Excellent	GP	Good-Excellent	$7{,}612Ø^{0.281}$	.87	4	118.5	114.2	15,157	Good-Excell
Sand	A-1-b	Good-Excellent	SP	Fair-Good	$1{,}831Ø^{0.366}$	.88	15	115.5	100.6	4,490	Poor-Fair
Sandy Clay	A-2-7	Fair-Good	SC	Poor-Fair	$4{,}609Ø^{0.208}$	.81	15	123.0	105.8	7,674	Fair

Table 13

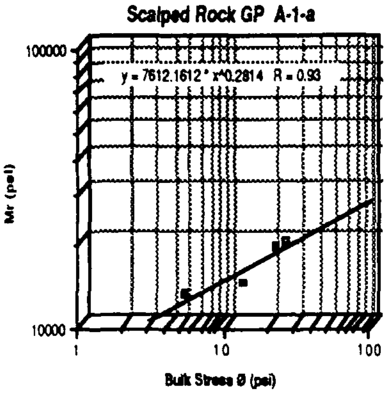


Figure 14 - Scalped Rock Results

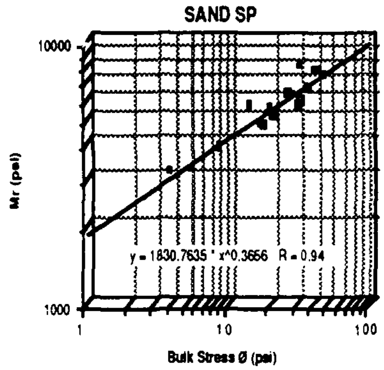


Figure 15 - Sand Results

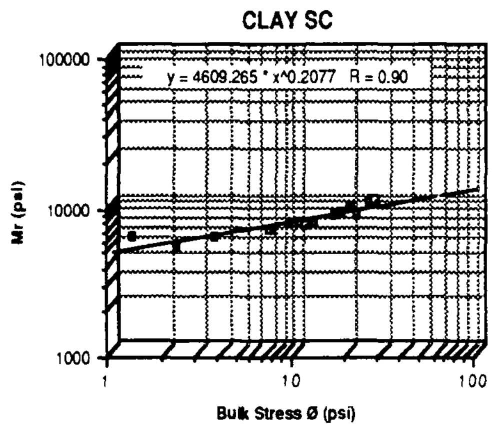


Figure 16 - Sandy Clay Results

All the samples demonstrated a much better correlation to bulk stress than to deviator stress as you would expect from the granular material, but not from the clay. This of course supports the position that unbound granular soils are sensitive to confining pressures. The sandy clay probably had a low enough percentage of clay to make it sensitive to confining. The sandy clay did have the flattest curve reaffirming that it was less sensitive than the other materials.

It can be concluded while "excellent" granular base course materials will perform well at high stresses, their high sensitivity to stress makes them only average subgrades where smaller principle stresses develop. This could be largely due to the lack of soil binder in the laboratory samples (AASHTO allows up to 50% > #40 sieve and 25% > #200). With

such a wide variety of materials satisfying the A-1 criteria, there is bound to be significant variability in stiffness. Our laboratory findings then did not support the AASHTO or the Unified Soil Classification System ratings of "excellent" for the unbound granular materials. The sandy clay soil performed better than might be expected in stiffness properties, but of course might be undesirable when swell and frost action are considered.

Fine-grained Soils

In an article by Boateng-Poku and Drumm (1989)[69] supporting a hyperbolic relationship between deviator stress and resilient moduli, a source of data was found for fine-grained soils. In their study, eleven soils representing a range of plasticity and strength properties were evaluated. The results obtained are shown in Table 14. Actual data was not available to evaluate the equation correlation coefficient (R^2).

In general, the fine-grained cohesive soils again performed better than would be expected based on stiffness criteria alone. On the other hand, the silty sands (SM) did not perform as well as expected. A significant in flux of water to the unstabilized silt or clay soil will drastically reduce these soil's elasticity and increase their plasticity making them unsuitable subgrades.

Local Subgrades - Washington Data

The most extensive and consistent data came from the Washington state highway system. The study, conducted by jointly by the civil engineering department of the University of Washington and the Washington State Department of Transportation materials laboratory, was established to create a new state overlay design procedure.[70] It included field sampling and laboratory tests at the test sites shown in Figure 17. Non-destructive testing with the Falling Weight Deflectometer was also conducted on the same sites and similar stiffness results were obtained adding to the level of confidence for the WA moduli. Table 15 organizes the results.

The Washington lab moduli are mostly consistent within classification groups and with index ratings as graphically demonstrated in Figures 18 &19:

COMPARISON OF LABORATORY RESILIENT MODULI TO SOIL INDEX
ON TENNESSEE HIGHWAYS

Location	AASHTO	Index Rating	USCS	Index Rating	Regression Equation	R^2	# Data Pts	Density (lb/ft3) Wet	Density (lb/ft3) Dry	Modulus @ d=7.01 psi	Stiffness Criteria Rating
Haywood	A-4(8)	Fair	CL	Poor-Fair	$\frac{(52.91+5.47d)}{d}$	-	-	122.1	103.5	13,018	Fair-Good
Haywood	A-6(24)	Poor	CL	Poor-Fair	$\frac{(36.74+8.57d)}{d}$	-	-	123.0	102.0	13,881	Fair-Good
Henderson	A-1-b(0)	Excellent	SM	Fair-Good	$\frac{(6.38+11.13d)}{d}$	-	-	128.1	111.7	12,040	Fair-Good
Benton	A-7-6(3)	Poor	ML	Poor-Fair	$\frac{(4.16+2.10d)}{d}$	-	-	108.8	83.7	2,693	Poor
Smith	A-4(12)	Fair	ML	Poor-Fair	$\frac{(60.97+8.64d)}{d}$	-	-	116.5	94.6	17,338	Good
Sumner	A-7-6(11)	Poor	CL	Poor-Fair	$\frac{(38.79+2.55d)}{d}$	-	-	121.0	96.8	8,084	Poor-Fair
Wilson	A-4(8)	Fair	CL	Poor-Fair	$\frac{(15.61+4.20d)}{d}$	-	-	128.0	109.0	6,427	Poor-Fair
Cumberland	A-4(5)	Fair	CL	Poor-Fair	$\frac{(29.55+4.70d)}{d}$	-	-	127.6	110.5	8,915	Fair
Campbell	A-2-4(0)	Good	SM-CL	Poor-Good	$\frac{(8.76+5.81d)}{d}$	-	-	130.0	114.7	7,060	Poor-Fair
Knox	A-7-6(28)	Poor	MH	Poor	$\frac{(55.4+3.71d)}{d}$	-	-	110.3	80.2	11,613	Fair-Good
Loudon	A-7-5(27)	Poor	MH	Poor	$\frac{(63.43+9.01d)}{d}$	-	-	116.4	88.3	18,059	Good

Table 14

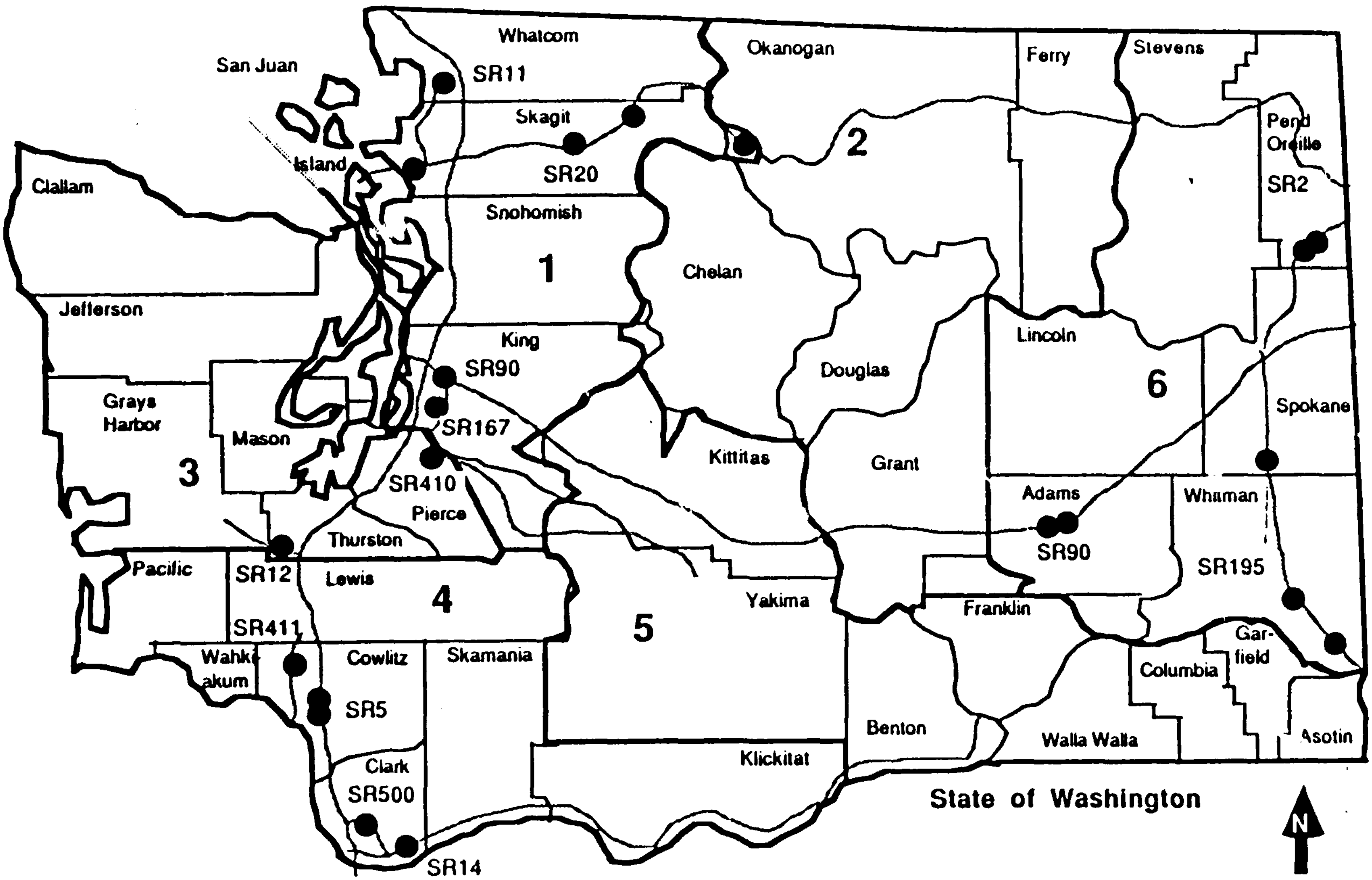


Figure 17 Location of Test Sections.

COMPARISON OF LABORATORY RESILIENT MODULI TO SOIL INDEX
ON WASHINGTON HIGHWAYS

Location	AASHTO	Index Rating	USCS	Index Rating	Regression Equation	R^2	# Data Pts	Density (lb/ft3) Wet	Density (lb/ft3) Dry	Modulus @ Ø=11.6 d=7.0 psi	Stiffness Criteria Rating
SR410 MP9.6	A-4(8)	Fair	ML	Poor-Fair	$3{,}194d^{0.358}$	.93	15	122.7	104.2	6,414	Fair
					$3{,}102Ø^{0.293}$	.64	20	125.0	106.5	6,361	Fair
					$4{,}511d^{0.210}$	.54	20	126.2	107.7	6,790	Fair
SR5 MP35.8	A-1-b(0)	Good-Excellent	SMd	Fair-Good	$8{,}132Ø^{0.317}$	.88	15	135.0	122.2	17,686	Excellent
					$10{,}240Ø^{0.323}$	.85	20	132.1	119.9	22,601	Excellent
					$16{,}420Ø^{0.207}$	.64	20	129.0	117.5	27,272	Excellent
SR500 MP3.2	A-1-a	Excellent	SMd	Fair-Good	$7{,}074d^{0.270}$	.90	15	144.0	130.3	11,968	Good
					$9{,}428d^{0.242}$	.84	20	143.3	131.0	15,104	Good
					$9{,}916Ø^{0.337}$	.85	20	139.5	130.4	22,650	Excellent
SR14 MP18.2	A-1-a	Excellent	GMd	Good-Excellent	$13{,}901d^{0.260}$	.88	15	132.2	122.3	23,064	Excellent
					$16{,}742Ø^{0.234}$	.74	20	131.1	121.2	29,709	Excellent
					$17{,}956Ø^{0.198}$	.68	20	137.1	126.2	29,172	Excellent
SR11 MP20.8	A-1-b	Good-Excellent	SMd	Fair-Good	$5{,}278Ø^{0.499}$	.83	15	135.7	127.9	17,932	Good-Excellent
					$7{,}629Ø^{0.418}$	.90	20	138.5	130.3	21,253	Excellent
					$8{,}977Ø0.373$	.88	20	140.0	131.2	22,396	Excellent
SR20 MP53.5	A-1-b	Good-Excellent	SP	Fair-Good	$5{,}278Ø^{0.531}$	.60	14	126.5	120.6	19,395	Excellent
					$6{,}693Ø^{0.511}$	.88	20	126.8	121.2	23,418	Excellent
					$5{,}172Ø^{0.595}$	.86	18	127.4	122.4	22,234	Excellent
SR20 MP77.5	A-1-a	Excellent	SMd	Fair-Good	$6{,}220Ø^{0.476}$	.95	15	130.0	124.8	19,974	Excellent
					$10{,}837Ø^{0.366}$	.88	20	132.9	127.2	26,577	Excellent
					$9{,}306Ø^{0.403}$	.90	20	131.7	125.6	24,988	Excellent

Table 15

COMPARISON OF LABORATORY RESILIENT MODULI TO SOIL INDEX
ON WASHINGTON HIGHWAYS (cont)

Location	AASHTO	Rating	USCS	Rating	Regression Equation	R^2	# Data Pts	Density (lb/ft3) Wet	Density (lb/ft3) Dry	Modulus @ Ø=11.6 d=7.0 psi	Criteria Rating
SR20 MP108.2	A-1-a	Excellent	GM	Good-Excellent	$10{,}991Ø^{0.353}$	.89	15	139.0	132.3	26,109	Excellent
					$11{,}849Ø^{0.361}$	.81	20	139.5	133.6	28,705	Excellent
					$10{,}579Ø^{0.375}$	.89	20	139.5	132.7	26,523	Excellent
SR20 MP140.8	A-1-a	Excellent	SMd	Fair-Good	$8{,}548Ø^{0.352}$	.91	15	131.4	126.8	20,256	Excellent
					$9{,}803Ø^{0.341}$	.83	20	134.7	129.5	22,612	Excellent
					$9{,}267Ø^{0.355}$	.87	20	132.6	127.4	22,122	Excellent
SR195 MP7.2	A-4(5)	Fair	ML	Poor-Fair	$18{,}049d^{-0.291}$	.56	15	134.8	118.1	10,241	Fair-Good
					$20{,}160d^{-0.247}$	.67	20	136.1	117.9	12,462	Fair-Good
					$16{,}019d^{-0.301}$	.89	20	136.1	117.9	8,914	Fair
SR195 MP20	A-4(8)	Fair	ML	Poor-Fair	$8{,}228d^{-0.179}$	.66	15	132.8	114.2	5,806	Poor-Fair
					$33{,}670d^{-0.275}$	.84	20	130.3	111.0	19,709	Excellent
					$23{,}037d^{-0.228}$	.83	20	131.6	113.5	14,778	Excellent
SR195 MP63.8	A-2-4	Good	SMu	Fair	$4{,}960d^{+0.079}$	.30	15	140.9	125.2	5,785	Poor-Fair
					$13{,}079d^{-0.184}$	.69	20	139.3	124.3	9,140	Fair
					$7{,}466d^{-0.091}$	.39	20	144.3	128.6	6,254	Poor-Fair
SR90 MP208.8	A-1-b	Good-Excellent	GMd	Good-Excellent	$14{,}934d^{0.228}$	.84	15	148.8	136.6	23,281	Excellent
					$24{,}899d^{0.181}$	.83	20	144.1	134.2	35,421	Excellent
					$22{,}540Ø^{0.193}$	.73	20	144.8	133.[illegible]	36,174	Excellent
									Avgerage	19,263	

Table 15 (cont)

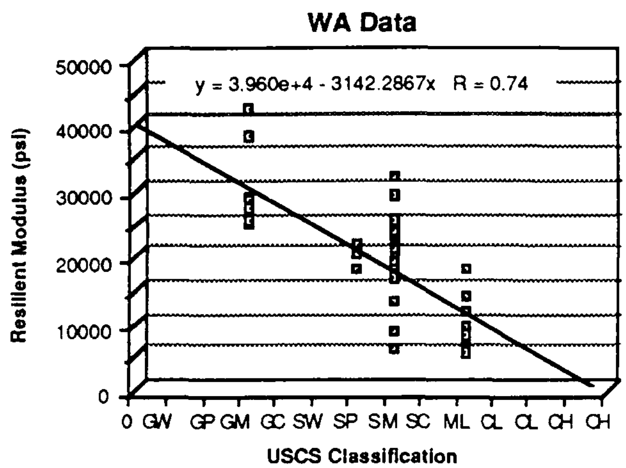


Figure 18 - USCS Classification and WA Mr Correlation

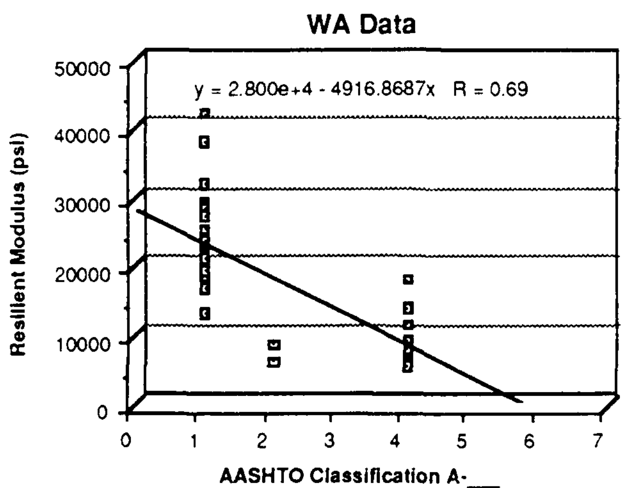


Figure 19 - AASHTO Classification and WA Mr Correlation

Washington is fortunate to have good quality natural A-1 soils (mostly gravels and consolidated silts) as opposed to the finer-grained subgrades of Tennessee. On average, the resilient moduli of subgrades on the 13 highway test sites equalled 19,263 psi.

Discussion - It is evident that much more data needs to be collected before definite conclusions can be drawn. However, some preliminary findings about soil moduli might be:

1. In general, soil stiffnesses are consistent with AASHTO and USCS index ratings for subgrade performance but not to the extent that a definitive range of moduli can be determined based solely on soil index (Table 16 & 17, Figures 20 & 21). The plot of moduli versus USCS classifications tends to produce a better fit than those of AASHTO. Generally all soil subgrade types fall between 2,000 and 30,000 psi.

Soil Type	# Data Pts	Mean Mr	Standard Deviation	97% Probability Range
A-1	32	21,321	7,611	6,099-36,543
A-2	5	7,139	1,297	4,545-9,733
A-3	-	-	-	Insufficient Data
A-4	13	10,552	4,576	1,400-19,704
A-5	-	-	-	Insufficient Data
A-6	1	-	-	Insufficient Data
A-7	4	10,113	6,443	0-23,000

Table 16 - Compiled Subgrade Moduli by AASHTO Index

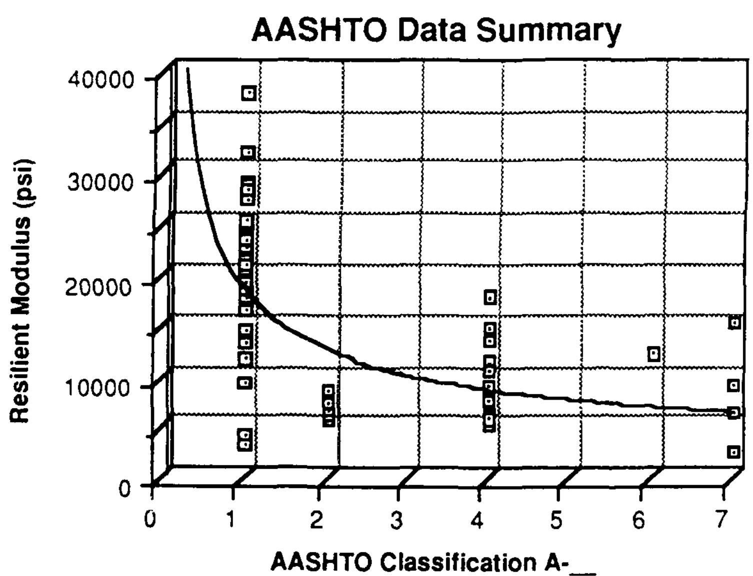


Figure 20 - Overall Subgrade Moduli by AASHTO Classification

Soil Type	# Data Pts	Mean Mr	Standard Deviation	97% Probability Range
GW	1	9,504	-	-
GP	1	14,552	-	-
GM	10	26,148	9,150	7,848-44,448
GC	0	-	-	-
SW	0	-	-	-
SP	4	17,324	8,879	0-35,082
SM	19	18,348	6,549	5,250-31,446
SC	1	7,456	-	-
ML	11	10,137	5,342	0-20,820
CL	5	10,051	3,211	3,629-16,473
OL	0	-	-	-
MH	2	14,836	-	-
CH	0	-	-	-

Table 17 - Compiled Subgrade Moduli by USCS Index

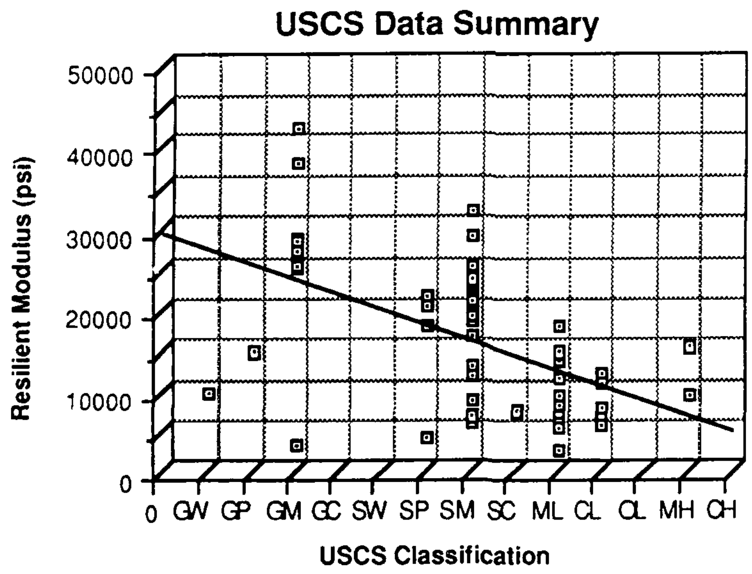


Figure 21 - Overall Subgrade Moduli by USCS Classification

2. The A-1 AASHTO classification covers a wide variety of material composition (with and without soil binders) which, in turn, produce a large range of resilient

moduli. The coarse granular soils without soil binder are very sensitive to confining pressures and can, therefore, sometimes be quite "soft" at the low pressures developed at subgrade depth.

3. The A-2 soils appeared to fall into a very tight stiffness group probably due to the fairly limited classification specifications and their relative insensitivity to moisture.

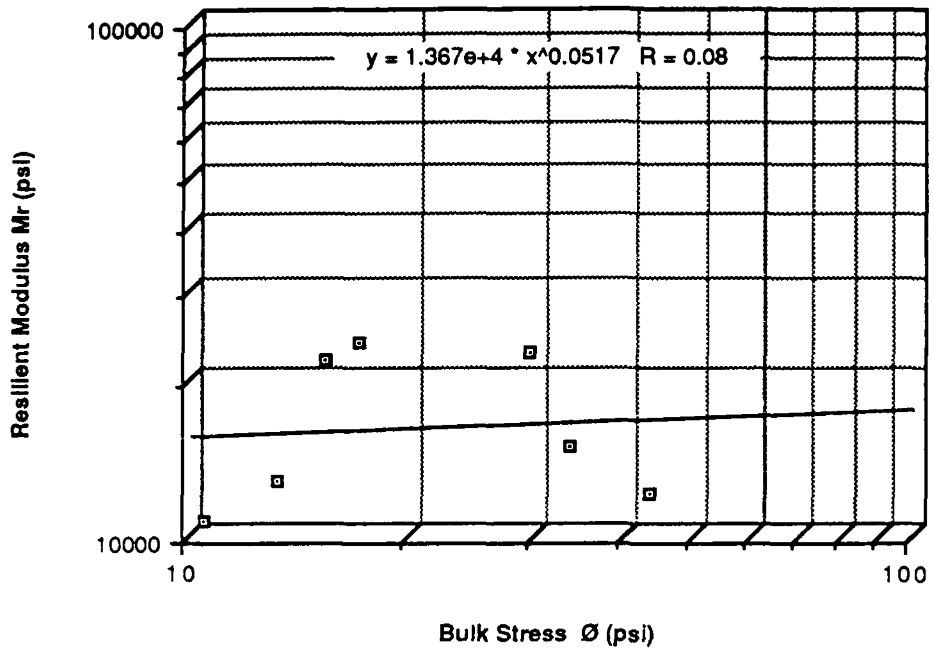
4. The moduli for A-4 to A-7 soils became less and less predictable which was probably related to the differing mineral content of each as well as the high sensitivity to moisture which was variable.

Although it was not possible to get a definitive range of resilient modulus values for each soil group index, a relative "feel" for the moduli you could gather from a given soil type was achieved. In a University of Illinois study, Thompson concluded that while "[Mr} is significantly correlated with liquid limit, plasticity index, group index, silt content, clay content, specific gravity, and organic carbon content" that "classifying the soil in the AASHTO, Unified, or USDA system does not place fine-grained soils into distinctive resilient behavior groups."[71] Our research supports Thompson's findings. In the absence of sample specific test results, it is apparent that moisture content and other factors <u>must</u> be considered when choosing resilient modulus values for design.

Resilient Modulus of CDF

<u>CDF 351</u> - The mix with only 30 lbs of cement supplied the results shown in Figures 22 and 23, and Table 18 (Samples A & B). Both cylinders tested exhibited a better regression fit with bulk stress than with deviator stress. This is not surprising since there is a minimal amount of cement in the mix. The equation correlation coefficient (R^2) was still very low, signifying that perhaps a different regression equation might better represent stabilized soils and CDF. Evaluating the moduli at a bulk stress of 11.6 psi (18k wheel load), an average value of 11,697 psi is obtained. This puts CDF 351 in the "Fair to Good" previously established "stiffness criteria rating".

CDF 351 Sample A



CDF 351 Sample A

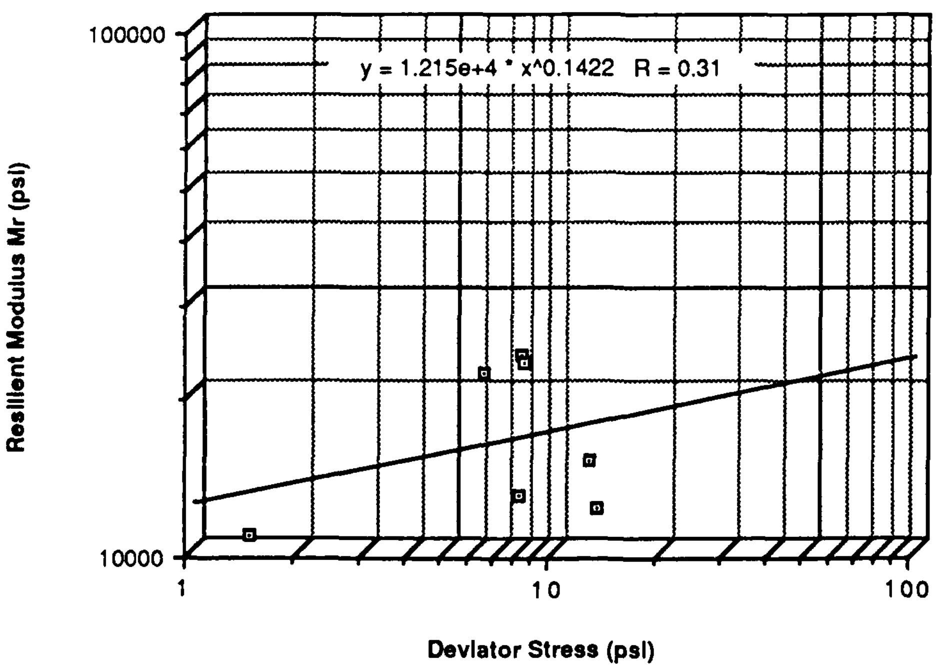
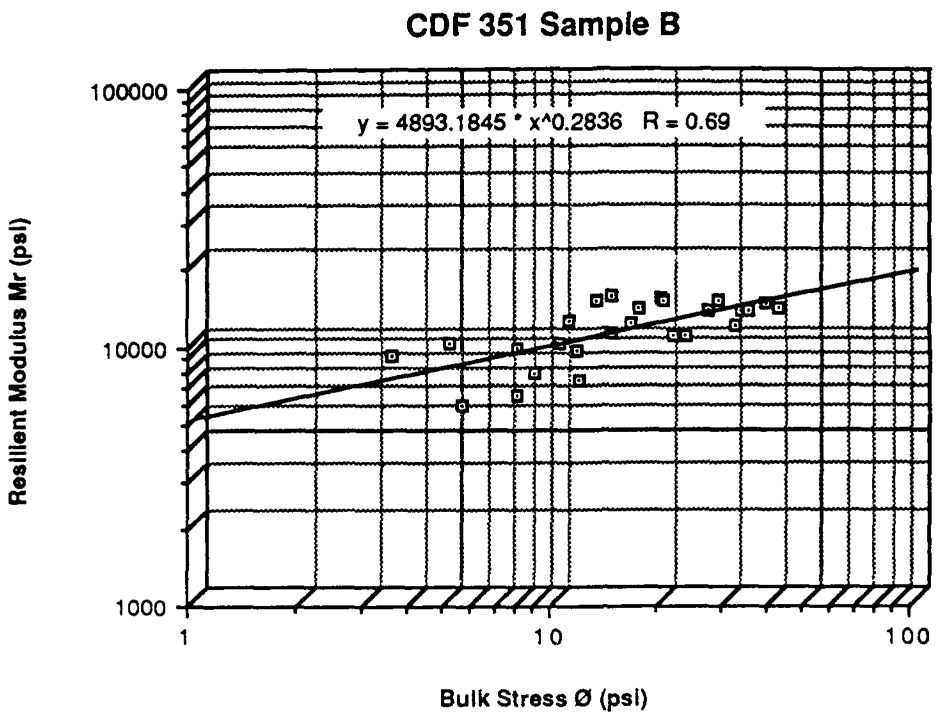


FIGURE 32


CDF 351 Sample B
y = 4893.1845 * x^0.2836 R = 0.69
100000
10000
1000
1
10
100
Resilient Modulus Mr (psi)
Bulk Stress Ø (psi)

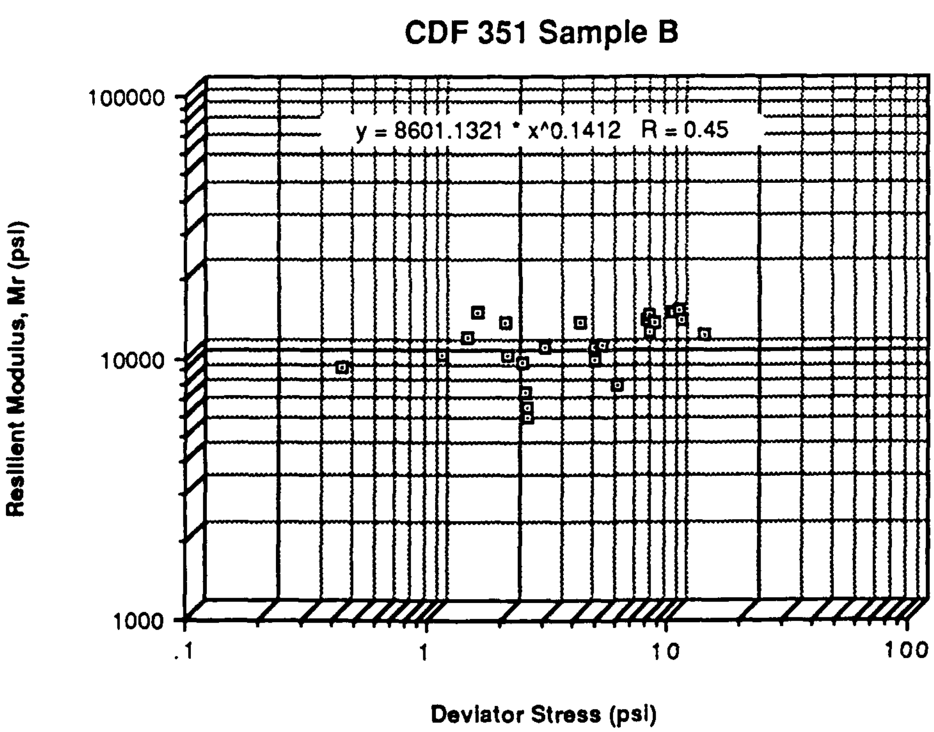

CDF 351 Sample B
y = 8601.1321 * x^0.1412 R = 0.45
100000
10000
1000
.1
1
10
100
Resilient Modulus, Mr (psi)
Deviator Stress (psi)

FIGURE 23

CDF LABORATORY RESILIENT MODULI SUMMARY

Test Date	CDF Sample #	Cement lb/CY	Fly Ash lb/CY	Aggregate lb/CY	Aggregate Type	Regression Equation	R^2	# Data Pts	Density (lb/ft3) Tested	Density (lb/ft3) Dry	w (%)	Modulus @ 18k load Ø=11.6,d=7.0	Stiffness Criteria Rating
12/18/89	A	30	300	2450	Bld Sand	$2,155Ø^{0.751}$	.59	5	-	-		13,579	Fair-Good
2/11/90	B	30	300	2450	Bld Sand	$4,893Ø^{0.284}$	.48	26	-	-	7.2	9,815	Fair-Good
				CDF	**351**	**Averages**	**.54**				**7.2**	**11,697**	**Fair-Good**
3/26/90	4C	40	0	2690	Sand	$1,932Ø^{0.123}$	.17	7	105.6	95.5	10.6	2,612	Poor
				CDF	**No Fly**	**Averages**	**.17**				**10.6**	**2,612**	**Poor**
4/26/90	1D	40	300	2585	Sand	$6,166d^{0.479}$	.86	16	116.3	103.4	12.5	15,671	Excellent
6/22/90	1C	40	300	2585	Sand	$32,360d^{0.209}$	.58	40	116.1	108.2	7.3	48,614	Excellent
7/5/90	1E	40	300	2585	Sand	$29,710Ø^{0.214}$	.71	16	115.6	104.2	10.9	50,199	Excellent
7/12/90	1H	40	300	2585	Sand	$12,250Ø^{0.425}$	.86	16	117.2	105.0	11.7	34,716	Excellent
7/13/90	1G	40	300	2585	Sand	$38,850d^{0.203}$	.55	15	116.3	104.9	10.9	57,686	Excellent
				CDF	**451**	**Averages**	**.71**		**116.3**	**105.1**	10.7	**41,377**	**Excellent**

RESILIENT MODULI AT RECOMPUTED STRESSES

CDF Sample	Modulus @18k load (Ø=20.7, d=9.2 psi)
1D	17,851
1C	51,456
1E	56,822
1H	44,404
1G	60,959
Average	46,298

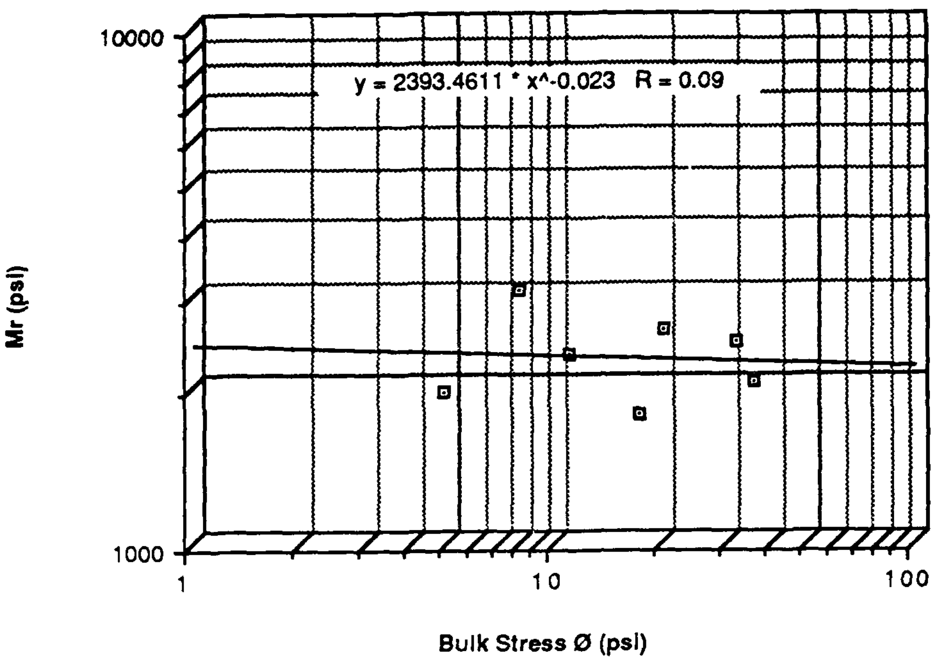
Table 18

CDF with no Fly Ash - As expected, the mix containing no fly ash was very weak obtaining an M_r of only 2,612 psi. The benefits of the fly ash filler can be seen in the lower density obtained in this mix. The absence of any pozzolanic reaction could have also contributed to its weakness. Figure 24 and Table 18 demonstrate the results (Sample 4C).

CDF 451 - The mix that the predominate amount of testing was done on proved to be an "excellent" subgrade by the "stiffness criteria rating" averaging 41,377 psi. The results for all five cylinders tested are shown in Figures 25 through 29 ,and Table 18. The moduli were obtained by using the 11.6 psi bulk stress and 7.0 psi deviator stress loads associated with the 18 kip equivalent single axle load in the best fit regression equation. Since these stresses were computed based on a 10,000 psi subgrade stiffness, they were somewhat inaccurate. Using the new 41k moduli and rerunning the ELSYM5 analysis produces bulk and deviator stresses of 20.7 and 9.2 psi respectively (See Figure 30 for calculations). The recomputed resilient modulus values are shown below the original values in Table 18. Sample 1D which had the lowest M_r value (15,671), also had the lowest dry density and highest tested moisture content. It is considered an outlier because it was the first sample tested from the batch and its membrane was found to be leaking, which could account for its better correlation coefficient with deviator stress. Excluding sample 1D and using the original figures, the standard deviation was 9,580 psi or 20%.

Sample 1C was tested over about a two month period in which it dried to about 7% moisture content. After a one month lapse in testing, the sample exhibited resilient modulus values averaging greater than 100,000 psi. Graphing the sample moisture content at the conclusion of testing vs. the resilient modulus values computed (Figure 31) shows that CDF is as sensitive to moisture conditions as is conventional subgrade materials.

CDF Sample 4C



CDF Sample 4C

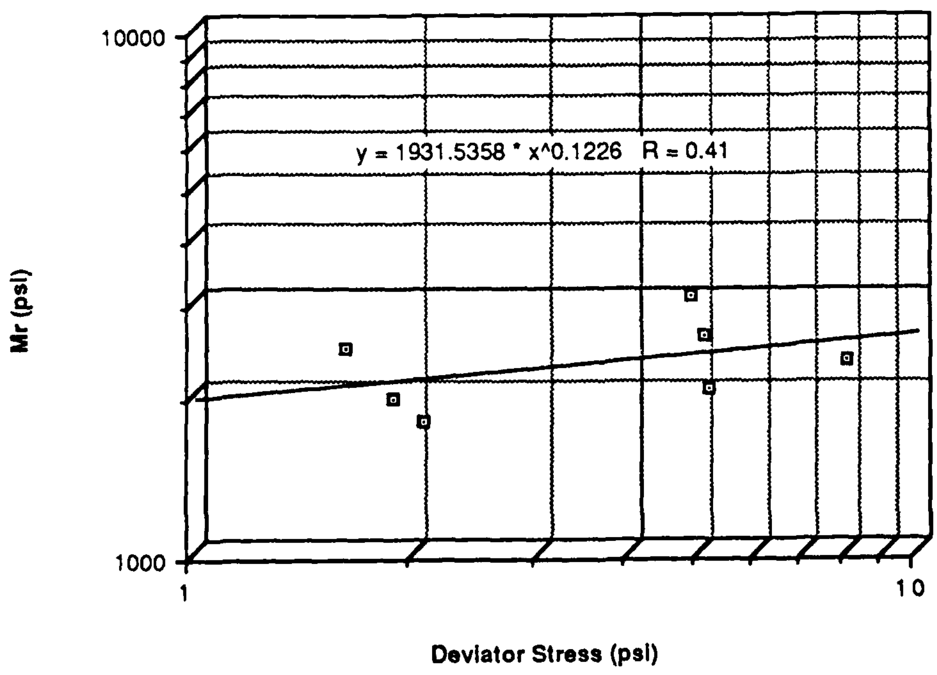
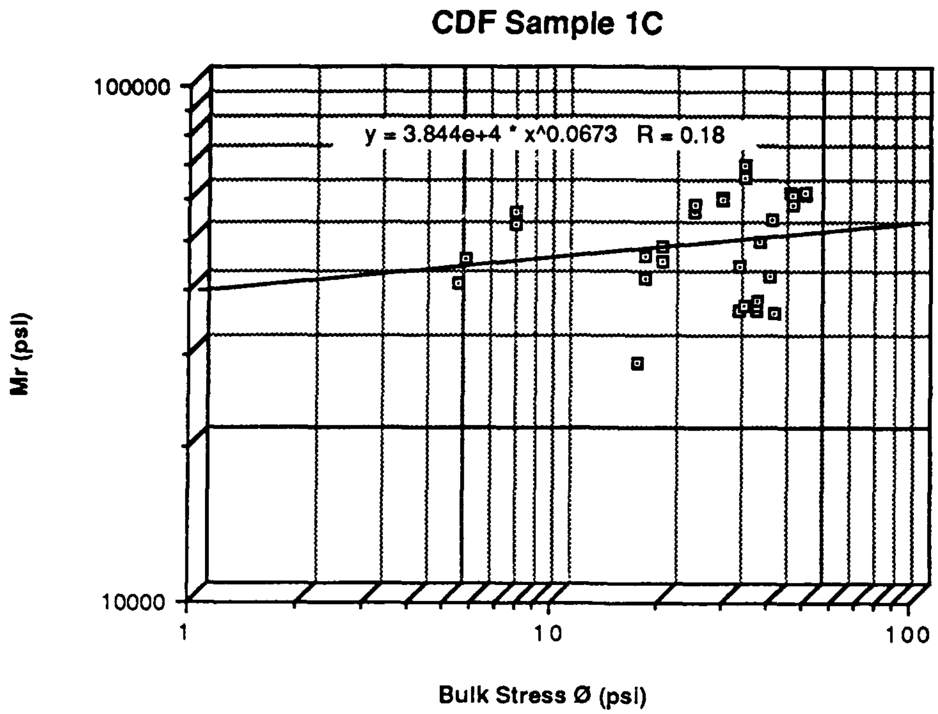
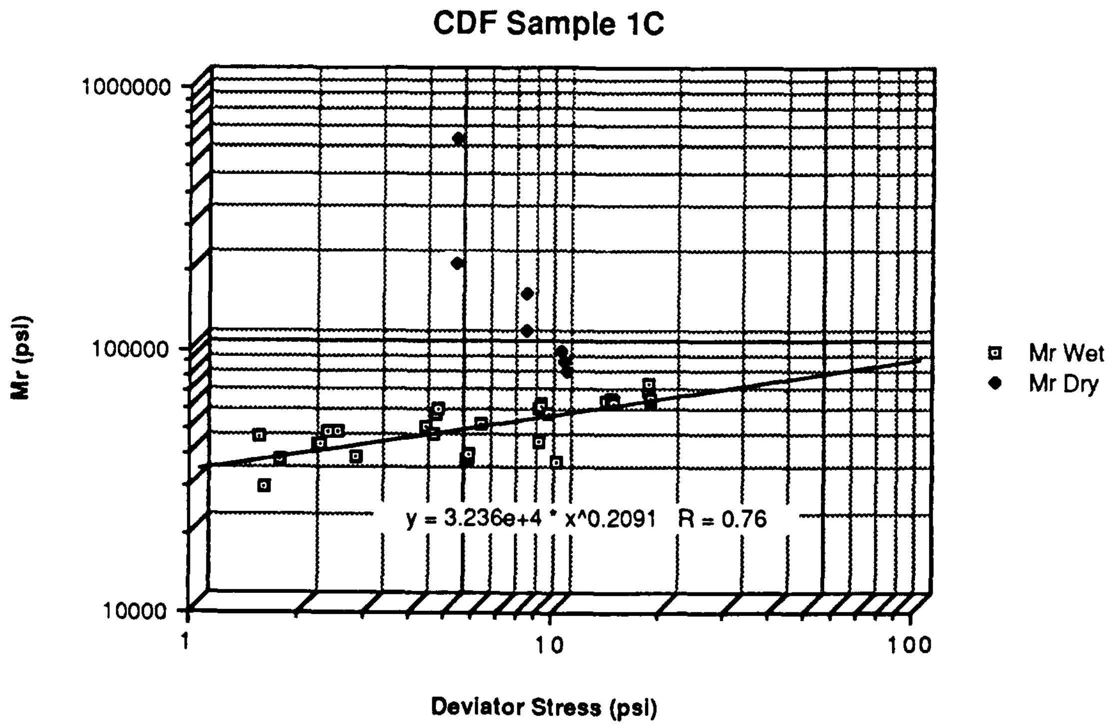


FIGURE 24

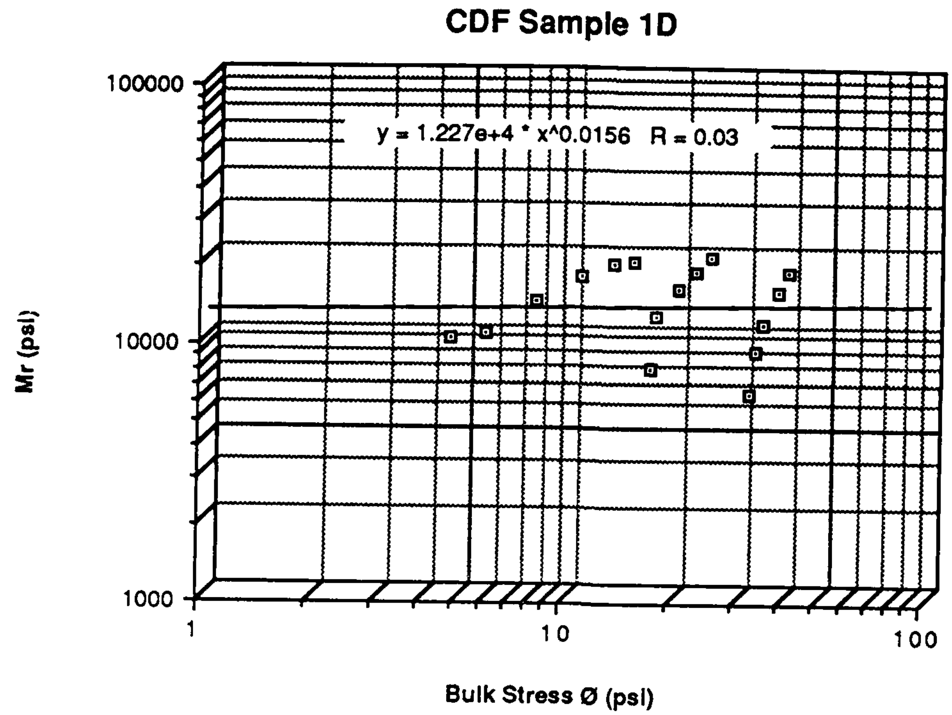


CDF Sample 1C
y = 3.844e+4 * x^0.0673 R = 0.18
100000
10000
1
10
100
Mr (psi)
Bulk Stress Ø (psi)



CDF Sample 1C
y = 3.236e+4 * x^0.2091 R = 0.76
1000000
100000
10000
1
10
100
Mr (psi)
Deviator Stress (psi)
Mr Wet
Mr Dry

FIGURE 25


CDF Sample 1D
y = 1.227e+4 * x^0.0156 R = 0.03
100000
10000
1000
Mr (psi)
1
10
100
Bulk Stress Ø (psi)

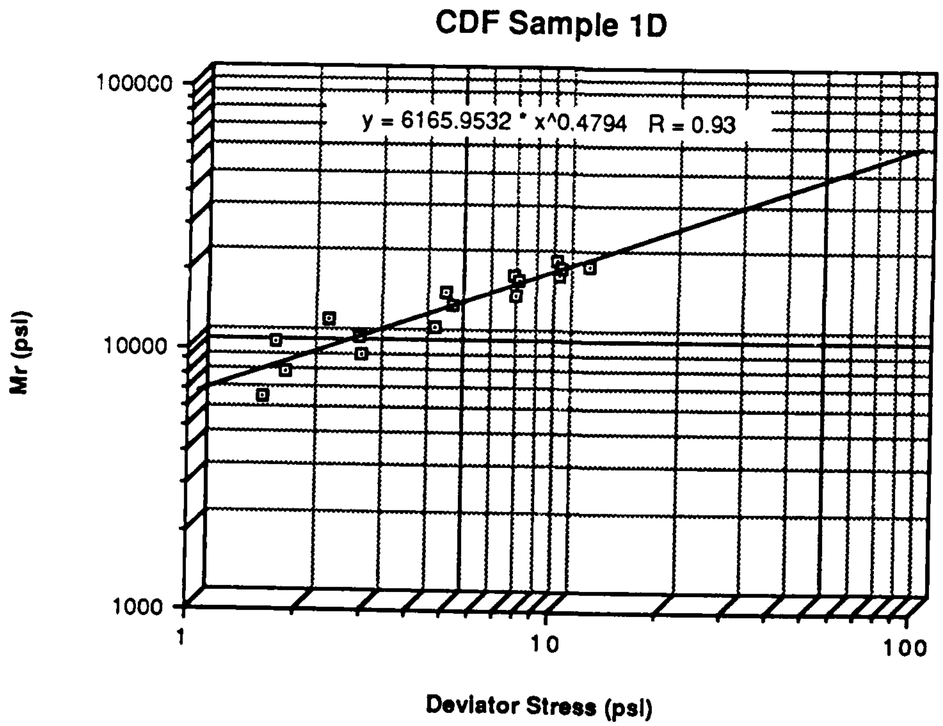
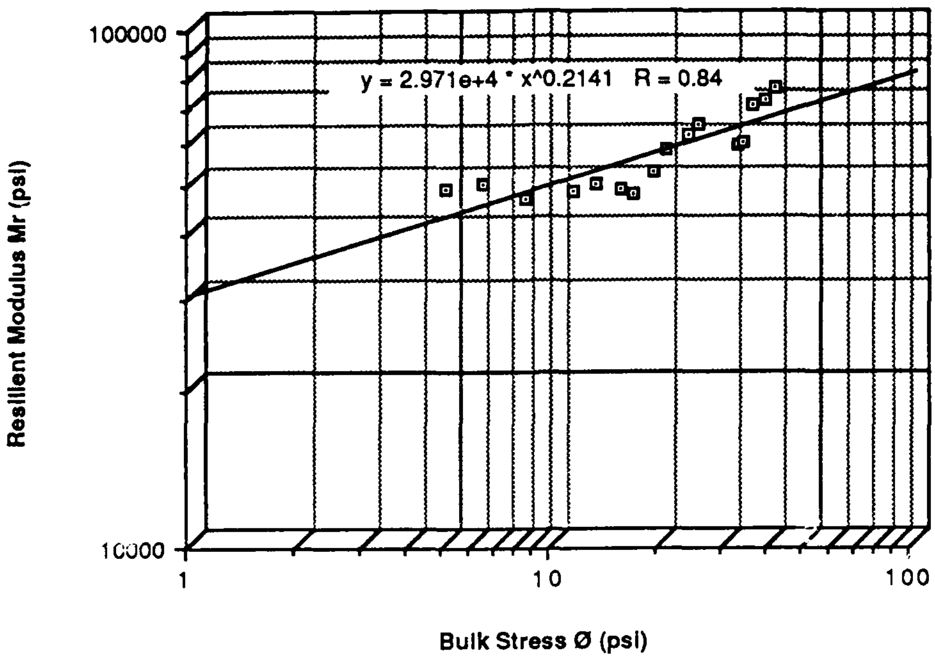

CDF Sample 1D
y = 6165.9532 * x^0.4794 R = 0.93
100000
10000
1000
Mr (psi)
1
10
100
Deviator Stress (psi)

FIGURE 26

CDF Sample 1E



CDF Sample 1E

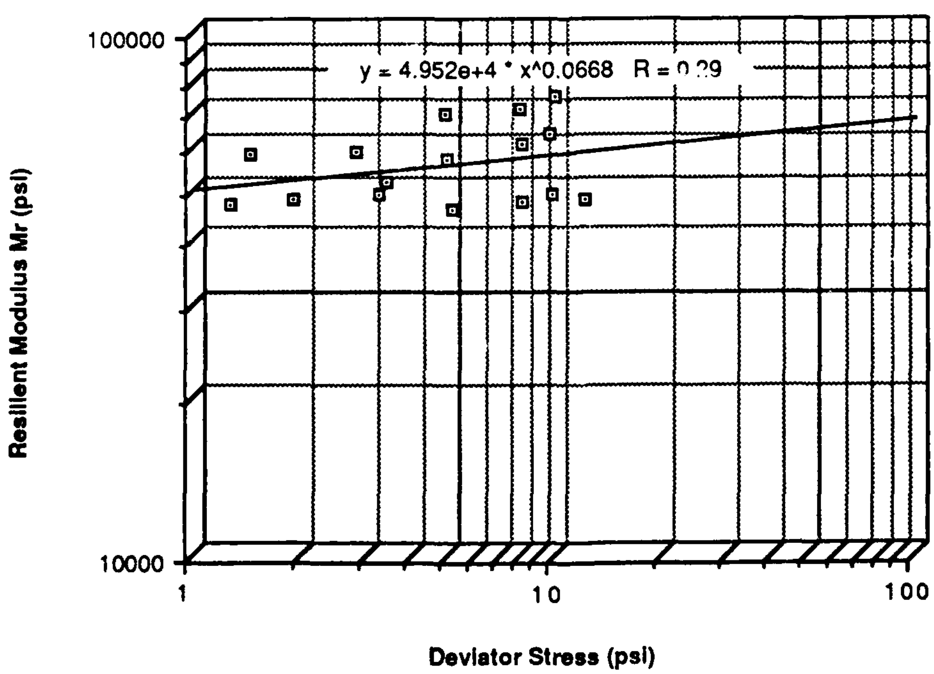
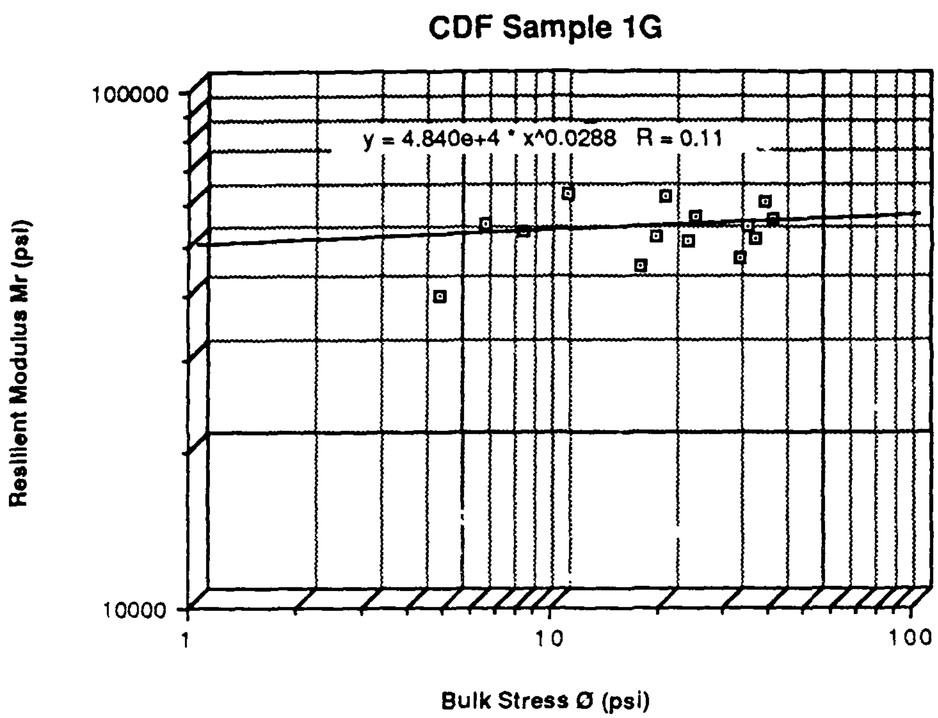


FIGURE 27


CDF Sample 1G
y = 4.840e+4 * x^0.0288 R = 0.11
Resilient Modulus Mr (psi)
100000
10000
1
10
100
Bulk Stress Ø (psi)

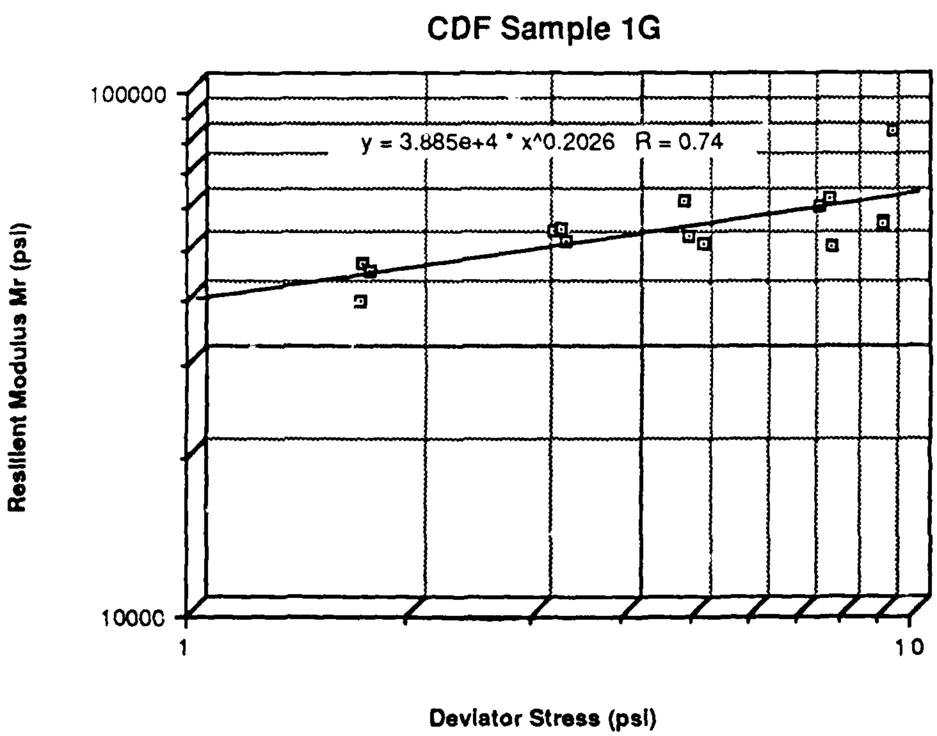
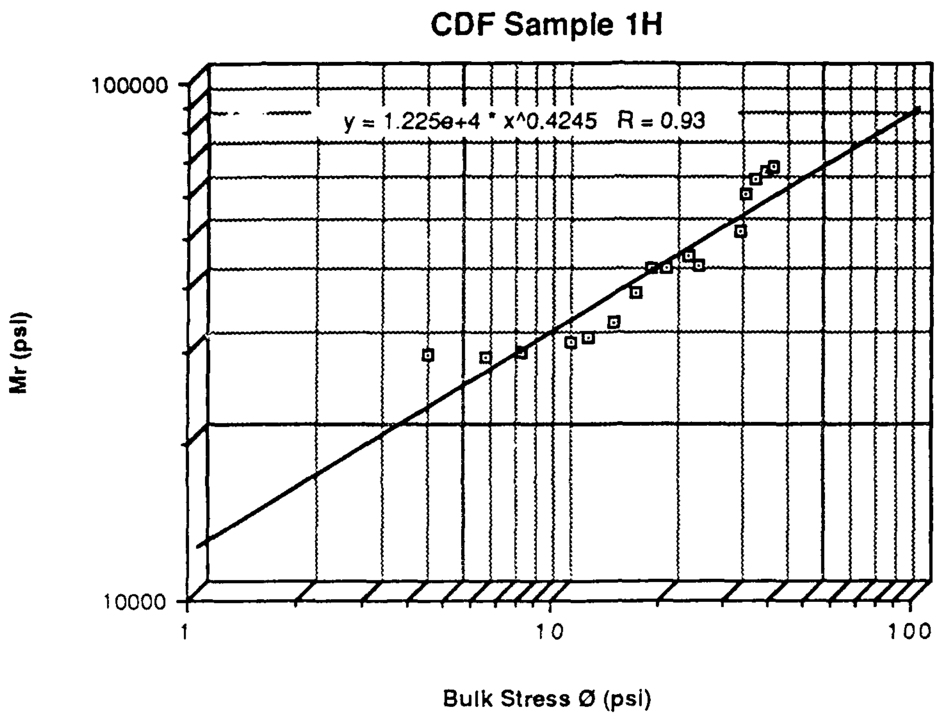

CDF Sample 1G
y = 3.885e+4 * x^0.2026 R = 0.74
Resilient Modulus Mr (psi)
100000
10000
1
10
Deviator Stress (psi)

FIGURE 28


CDF Sample 1H
y = 1.225e+4 * x^0.4245 R = 0.93
100000
10000
1
10
100
Mr (psi)
Bulk Stress Ø (psi)

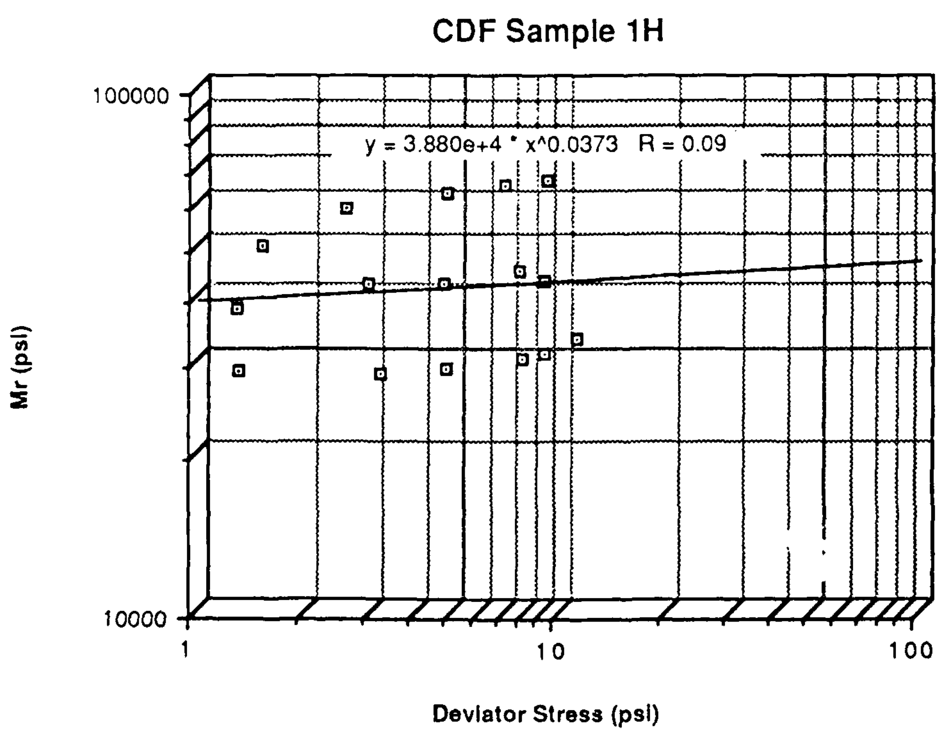

CDF Sample 1H
y = 3.880e+4 * x^0.0373 R = 0.09
100000
10000
1
10
100
Mr (psi)
Deviator Stress (psi)

FIGURE 29

ELASTIC SYSTEM -- [redacted]

LAYER	ELASTIC MODULUS	POISSONS RATIO	THICKNESS
1	400000.	.350	6.000 IN
2	10500.	.400	6.000 IN
3	[redacted]	.400	SEMI-INFINITE

ONE LOAD(S), EACH LOAD AS FOLLOWS

TOTAL LOAD..... 9000.00 LBS
LOAD STRESS.... 100.00 PSI
LOAD RADIUS.... 5.35 IN

LOAD	LOCATED AT X	Y
1	.000	.000

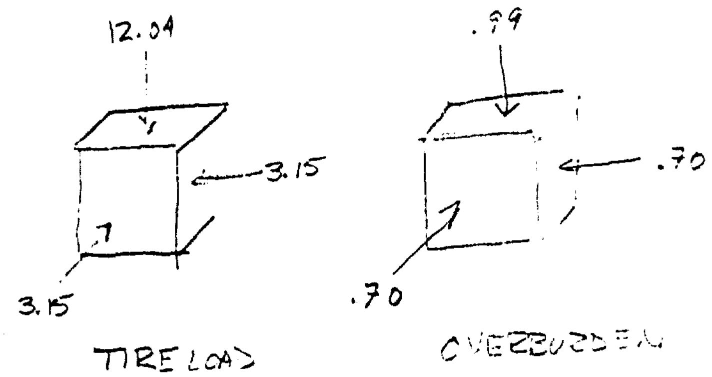
RESULTS REQUESTED FOR SYSTEM LOCATION(S)

DEPTH(S)
Z= 12.10
X-Y POINT(S)

X	Y
.00	.00

Z= 12.10 LAYER NO. 3

X	Y
.00	.00



NORMAL STRESSES
SXX -.3149E+01
SYY -.3149E+01
SZZ -.1204E+02

SHEAR STRESSES
SXY .0000E+00
SXZ .0000E+00
SYZ .0000E+00

$\sigma_1 = 12.04 + .99 = 13.03$ psi
$\sigma_2 = 3.15 + .70 = 3.85$
$\sigma_3 =$ " " $= 3.85$
$\theta = 20.73$ psi

$\sigma_d = 13.03 - 3.85 = 9.18$ psi

PRINCIPAL STRESSES
PS 1 -.3149E+01
PS 2 -.3149E+01
PS 3 -.1204E+02

PRINCIPAL SHEAR STRESSES
PSS 1 .4444E+01
PSS 2 .0000E+00
PSS 3 .4444E+01

DISPLACEMENTS
UX .0000E+00
UY .0000E+00

FIGURE 30

ELASTIC SYSTEM - [redacted]

LAYER	ELASTIC MODULUS	POISSONS RATIO	THICKNESS
1	400000.	.350	6.000 IN
2	10500.	.400	6.000 IN
3	[redacted]	.400	SEMI-INFINITE

ONE LOAD(S), EACH LOAD AS FOLLOWS

TOTAL LOAD..... 45113.00 LBS
LOAD STRESS.... 1.00 PSI
LOAD RADIUS.... 120.00 IN

LOAD	LOCATED AT X	Y
1	.000	.000

RESULTS REQUESTED FOR SYSTEM LOCATION(S)

DEPTH(S)
Z= 12.10
X-Y POINT(S)

X	Y
.00	.00

Z= 12.10 LAYER NO. 3

X	Y
.00	.00

NORMAL STRESSES
SXX -.6950E+00
SYY -.6950E+00
SZZ -.9897E+00

SHEAR STRESSES
SXY .0000E+00
SXZ .0000E+00
SYZ .0000E+00

PRINCIPAL STRESSES
PS 1 -.6950E+00
PS 2 -.6950E+00
PS 3 -.9897E+00

PRINCIPAL SHEAR STRESSES
PSS 1 .1474E+00
PSS 2 .0000E+00
PSS 3 .1474E+00

DISPLACEMENTS
UX .0000E+00
UY .0000E+00

FIGURE 30 (cont)

Effect of Moisture on CDF's Stiffness

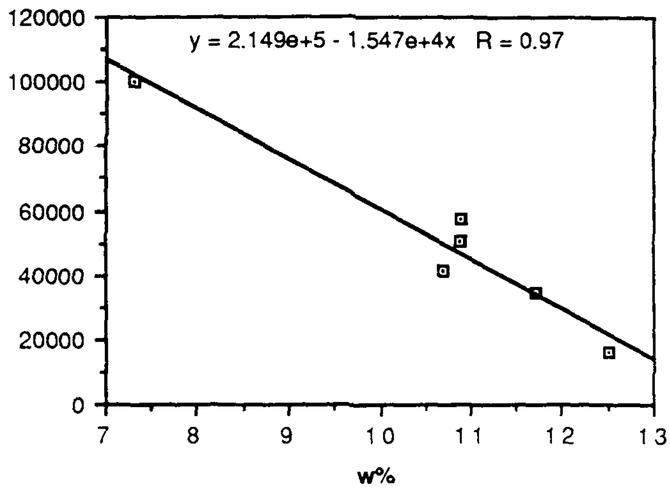


Figure 31

CONCLUSIONS & RECOMMENDATIONS

1. Resilient modulus for soil subgrades evaluated on Washington state highways averaged 19,300 psi. Established criteria gives Washington subgrade materials an "excellent" rating. From all indications CDF exhibits stiffnesses well in excess of typical pavement subgrades. Resilient modulus values in the 40 ksi range will provide flexible pavements with a strong foundation. The CDF mix could be adjusted to be less stiff by reducing the amount of cement slightly to be more cost efficient. It appears a cement content of about 35 lbs / CY might be sufficient, however, there appears to be a rapid transition in weakening stiffnesses using cement contents between 40 and 30 lbs/CY.

2. Fatigue and durability problems were not encountered when subjecting the specimen to 612,000 equivalent single axle loads. A conservative analysis had plastic strain at 0.38% after as many loads. Additional studies could be done to look solely at this query.

3. Moisture content plays a role in the stiffness of the CDF as well as conventional subgrades. When a cylinder was allowed to dry to 7% moisture, it stiffened significantly. Water content had a direct correlation on the resulting stiffness.

4. Before placing CDF above the frost line, it should be tested to determine its freeze-thaw susceptibility.

5. Fly ash plays an important role in achieving proper density and resilient modulus results. When a CDF sample containing no fly ash was tested, it had a very low density and produced very weak stiffness results.

6. Surprisingly, decent correlation coefficients using conventional backfill regression equations were obtained. However, regression equations representing CDF's resilience were sometimes a function of deviator stress and other times a function of bulk stress. Additional study is needed to determine a regression equation which best represents M_r as a function of stresses. Certainly, the more cement content in the mix, the better the equation will fit a deviator model.

7. As long as freeze-thaw and drainage are considered for the individual application, Controlled Density Fill is a viable alternative to conventional backfill pavement subgrades in utility trenches.

ACKNOWLEDGEMENTS

This research study was conducted with the help of Pozzolanic, Stoneway Concrete, Associated Sand and Gravel, the City of Everett, WA and the Federal Highway Administration. The assistance of Jenny Flechsig at Pozzolanic and Dr. Don Janssen at the University of Washington was especially appreciated.

REFERENCES

---

[1]Tom H. Johnston, "Restoring Utility Cuts," *APWA Reporter*, August 1986, p.12.

[2]Al Goldberg, "Kuhlman Corp. and Detroit Edison Concoct a Formula, Making Fly Ash Settle Down To A New Role, Product A Help To Contractors," *The Blade*: Toledo Ohio, 21 July 1974.

[3]Pozzolanic, Island Corporate Center, 7525 SE 24th St, Mercer Island, WA 98040, Tom Fox (Manager Technical Services), 1-800-426-5171.

[4]Stoneway Concrete, 1915 Maple Valley Highway, Renton, WA 98055, Mel Hitch (Sales Representative), (206) 226-1000.

[5]Associated Sand & Gravel Company, Inc. Box 2037/6300 Glenwood Ave., Everett, WA 98203, Tom Howerton (Sales Manager), (206) 348-6354.

[6]Pozzolanic Northwest, Inc. advertisement, Mercer Island, WA.

[7]William E. Brewer, "The end of the backfill problem, Controlled-density fill," *Concrete Construction*, Oct. 1975, p. 448.

[8]"Fly ash gains, with feds' aid", *Engineering News Record*, May 2, 1985, p. 13.

[9]Jenny L. Flechsig, Technical Services Representative, Pozzolanic

[10]"Fly ash gains, with feds' aid", *Engineering News Record*, May 2, 1985, p. 13.

[11]"The best thing to hit the road since the wheel!" *Inland/Tilbury inform*, Inland Cement/Tilbury Cement, Vol. 12, No. 2,. Summer 1987, p.2.

[12]"Development of a Controlled Density Fill For Use in Salt Lake City, Utah." Pozzolanic International, Mercer Island, WA, 1982.

[13]Mike Weitkamp, Maintenance and Operations Supervisor, City of Everett Public Works Dept. - Street Division.

[14]Dr. Don Janssen, CETS 564 unpublished class notes, University of Washington, Seattle, 1990.

[15]*The Unified Soil Classification System. Appendix B, Characteristics of Soil Groups Pertaining to Roads and Airfields* (Vicksburg, MS: U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, 1957)

[16]*The Unified Soil Classification System, Appendix B, Characteristics of Soil Groups Pertaining to Roads and Airfields* (Vicksburg, MS: U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, 1957) Table B1.

[17]*Soils Manual for the Design of Asphalt Pavement Structures, 4th ed.* , (College Park, MD: The Asphalt Institute, 1986) p. 42.

[18]AASHTO Designation; T 274-82 , "Standard Method of Test for Resilient Modulus of Subgrade Soils," (1986). American Association of State Highway and Transportation Officials, Washington, DC.

[19]Marshall R. Thompson, "Factors Affecting the Resilient Moduli of Soils and Granular Materials," presented at Workshop on Resilient Modulus Testing, Oregon State University, Corvallis, OR, March 28-30, 1989.

[20]M.R. Thompson and Q.L. Robnett, "Resilient Properties of Subgrade Soils," *Journal of the Transportation Engineering Division*, ASCE, Vol. 105, pp. 71-89.

[21]Yaw Boateng-Poku and Eric C. Drumm, "Hyperbolic Model for the Resilient Modulus Response of Fine-Grained Subgrade Soil," *Resilient Moduli of Soils: Laboratory Conditions*, Geotechnical Special Publication #24, ASCE, Oct. 10, 1989, p. 2.

[22]Yaw Boateng-Poku and Eric C. Drumm, "Hyperbolic Model for the Resilient Modulus Response of Fine-Grained Subgrade Soil," *Resilient Moduli of Soils: Laboratory Conditions*, Geotechnical Special Publication #24, ASCE, Oct. 10, 1989, p. 2.

[23]J.P. Mahoney, R.G. Hicks, N.C. Jackson, *Flexible Pavement Design and Rehabilitation*, March 1988, p. 57.

[24]*Handbook of Chemistry and Physics*, 58th Ed. (Palm Beach, FL: CRC Press, Inc., 1978).

[25]Michael R. Lindeburg, *Civil Engineering Reference Manual*, 4th Ed. (San Carlos, CA: Professional Publications, Inc., 1986), pp. 12-3.

[26]Robert J. Hoyle, *Wood Technology in the Design of Structures*, 4th Ed. (Missoula, MT: Mountain Press Publishing Co., 1978), p. 21.

[27]*AASHTO Guide for Design of Pavement Structures* (Washington D.C.: American Association of State Highway and Transportation Officials, 1986), p. I-14.

[28]*AASHTO Guide for Design of Pavement Structures* (Washington D.C.: American Association of State Highway and Transportation Officials, 1986), p. I-14.

[29]J.M. Moossazadeh and W. Witczak (1981), "Prediction of Subgrade Moduli for Soil That Exhibits Nonlinear Behavior," *Transportation Research Record*, No. 810, pp. 9-17.

[30]Yaw Boateng-Poku and Eric C. Drumm, "Hyperbolic Model for the Resilient Modulus Response of Fine-Grained Subgrade Soil," *Resilient Moduli of Soils: Laboratory Conditions*, Geotechnical Special Publication #24, ASCE, Oct. 10, 1989, p. 2.

[31]Joe P. Mahoney, "Regression Analysis For WSDOT Material Applications," Washington State Transportation Center and Washington State Dept. of Transportation, Feb. 1988.

[32]D.V. Lasater & V. Turner, Unpublished paper and research conducted for CETS 564 - Soil & Site Improvement, University of Washington, Seattle, WA, Winter quarter 1990.

[33]J.A. Lary, J.P. Mahoney, C.A. Bell and R.G. Hicks, "Seasonal Environmental Effect on the Strength of Pavement Structures," Research Report for the U.S. Forest Service, University of Washington, Seattle, May 1983.

[34]*AASHTO Guide for Design of Pavement Structures* (Washington D.C.: American Association of State Highway and Transportation Officials, 1986), p. II-21.

[35]Mohamed K. Elfino and John L. Davidson, "Modeling Field Moisture in Resilient Moduli Testing," *Resilient Moduli of Soils: Laboratory Conditions* (New York: American Society of Civil Engineers, 1989) pp. 31-51.

[36]"Mechanistic-Based Overlay Design Procedure for Washington State Flexible Pavements," WA-RD 170.1, Washington State Department of Transportation, Planning, Research and Public Transportation Division, January 1989, p.43.

[37]Mahoney, J.P., Jackson, N.C., and Hicks, R.G., "Flexible Pavement Design and Rehabilitation", 1988. p. 29.

[38]From AASHTO at 10 psi (CBR X 250 = M_r).

[39]*The Unified Soil Classification System, Appendix B, Characteristics of Soil Groups Pertaining to Roads and Airfields* (Vicksburg, MS: U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, 1957) Table B1.

[40]Dr. Don Janssen, Unpublished notes from CETS 464 - Construction Materials II, University of Washington, Seattle, WA, Fall quarter 1989.

[41]Steven H. Kosmatka and William C. Panarese, *Design and Control of Concrete Mixtures*, (Skokie, Illinois: Portland Cement Association, 1988), p. 30.

[42]Mahoney, J.P., Jackson, N.C., and Hicks, R.G., "Flexible Pavement Design and Rehabilitation", 1988.

[43]David C. Esch and Robert L. McHattie, "Prediction of Roadway Strength From Soil Properties," *Transportation Research Record, 918*, pp.30-34.

[44]Ronald D. Bernard & Roger S. Tansley, "A Laboratory Testing Program for Development of a Lean Mix Backfill Specification," prepared for the Dept. of Housing and Urban Development, October 1981, p. 9.

[45]Dr. Don Janssen, Unpublished notes from CETS 424 - Pavement Design, University of Washington, Seattle, Fall quarter 1989.

[46]Dr. Don Janssen, Unpublished notes from CETS 464 - Construction Materials II, University of Washington, Seattle, Fall quarter 1989.

[47]Steven H. Kosmatka and William C. Panarese, *Design and Control of Concrete Mixtures*, (Skokie, Illinois: Portland Cement Association, 1988), p. 15-21.

[48]Steven H. Kosmatka and William C. Panarese, *Design and Control of Concrete Mixtures*, (Skokie, Illinois: Portland Cement Association, 1988), p. 68-9.

[49]Steven H. Kosmatka and William C. Panarese, *Design and Control of Concrete Mixtures*, (Skokie, Illinois: Portland Cement Association, 1988), p. 68.

[50]Steven H. Kosmatka and William C. Panarese, *Design and Control of Concrete Mixtures*, (Skokie, Illinois: Portland Cement Association, 1988), p. 68.

[51]"Ash Utilization in Roadways, Embankments and Backfills," *EPRI Quarterly Report*, Volume 6, Number 4, November 1989, p.12.

[52]"Flowable Fill, A New Product = A New Market," *Promotion Pointers*, National Ready Mixed Concrete Association, Number 273, July 1985, p.2.

[53]Stoneway Concrete, CDF 351 Mix, Renton, WA.

[54]Memorandum 1306-07 Appendices, Kenmore Interceptor Predesign, to Martin C. Dirks, Technical Services Director Kenmore Interceptor Design Team, dated Jan 15, 1985.

[55]"$2.1 Million Mine Subsidence Control Project Initiated in Fairmont - Fly Ash / Cement Grout To Be Used," *Ash at Work* Newspaper, 1983, p. 4.

[56]Bob Drake, "Controlled Density Fill gaining acceptance," *Concrete Producer News*, May 1989, p. 52.

[57]"Ready mixed concrete products accelerate construction," *Ohio Paver*, Feb. 1986, p. 7.

[58]Letter from Oliver T. Harding, State of WA DOT, Project Engineer for I-90 Mount Baker Ridge Tunnel, dated August 15, 1984.

[59]Iowa Dept. of Transportation, "Supplemental Specifications for Flowable Mortar," Specification #957, April 10, 1984.

[60]City of Everett, WA, Dept. of Public Works, Standard Specification 9-30.10.

[61]City of Selah, WA, Section 37 of Standard Specifications for Street Construction revision of Sep. 10, 1984 (Lean Mix Cement Concrete).

[62]Chris Bedford, "Control density fill tested for ability to flow & give proper bearing, excavation ease & strength," *Intermountain Contractor*, Dec. 15, 1986, p. 8.

[63]Pozzolanic International records.

[64]Steven H. Kosmatka and William C. Panarese, *Design and Control of Concrete Mixtures*, (Skokie, Illinois: Portland Cement Association, 1988), pp. 123-9.

[65]6" AC Layer with E_{ac} = 400,000 psi, 6" Base Course Layer with E_{bs} = 10,500 psi, and Subgrade with M_r = 10,000 psi.

[66]J.P. Mahoney, "State-of-the-Art on Asphalt Concrete Overlay Design Procedures," University of Washington, p. 108.

[67]Darin V. Lasater, Unpublished report for CETS 424 - Pavement Design, University of Washington, Seattle, Fall quarter 1989.

[68]"Asphalt Overlays for Highway and Street Pavement Rehabilitation," Manual Series No. 17, The Asphalt Institute, College Park, MD, June 1983.

[69]Yaw Boateng-Poku and Eric C. Drumm, "Hyperbolic Model for the Resilient Modulus of Fine-Grained Subgrade Soil," *Resilient Moduli of Soils: Laboratory Conditions* (New York: American Society of Civil Engineers, 1989) pp. 1-15.

[70]Mahoney, Lee, Jackson, Newcomb, *Mechanistic-Based Overlay Design Procedure for Washington State Flexible Pavements* (Olympia, WA: Washington State Department of Transportation & Washington State Transportation Center, 1989), pp. 1-8.

[71]Marshall R. Thompson, "Factors Affecting the Resilient Moduli of Soils and Granular Materials," presented at Workshop on Resilient Modulus Testing, Oregon State University, Corvallis, OR, March 28-30, 1989.